DTIC FILE COPY

CRREL

AD-A224 072

SPECIAL REPORT 90-12

(4)

Use of Soft Grade Asphalts in Airfields and Highway Pavements in Cold Regions

Vincent Janoo

May 1990

Special Report 90-12

U.S. Army Corps of Engineers
Cold Regions Research & Engineering Laboratory

Use of Soft Grade Asphalts in Airfields and Highway Pavements in Cold Regions

Vincent Janoo

May 1990

Prepared for
OFFICE OF THE CHIEF OF ENGINEERS
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL AVIATION ADMINISTRATION

PREFACE

This report was prepared by Dr. Vincent Janoo, Research Civil Engineer, Civil and Geotechnical Engineering Research Branch, Experimental Engineering Division, U.S. Army Cold Regions Research and Engineering Laboratory. Funding for this research was provided by the DA Facilities Investigation and Studies Program, Work Unit 104, *Field Performance of Soft Grades of Asphalt Pavements;* by DA Project 4A762784AT42, *Design, Construction and Operations Technology for Cold Regions,* Task BS, Work Unit 003, *Asphalt Pavements in Cold Regions;* and by the U.S. Department of Transportation, Federal Aviation Administration, under Interagency Agreement DTFA-01-84-2-02038.

The author thanks Dr. Richard Berg, William Quinn and Robert Eaton for technically reviewing this report. Special thanks are expressed to John Bayer, Jr., and Wendy Allen for their assistance in conducting the field survey.

The contents of this report are not to be used for advertising or promotional purposes. Citation of trade names does not constitute an endorsement or approval of the use of such commercial products.

CONTENTS

ILLUSTRATIONS

Figure

ILLUSTRATIONS (cont.)

Figure

TABLES

Table

Use of Soft Grade Asphalts in Airfields and Highway Pavements in Cold Regions

VINCENT JANOO

INTRODUCTION

In cold regions, airfield and highway pavements are subjected to cyclic thermal loading, as well as traffic loading. Pavement temperatures can range from –50°C in the winter to 60°C in the summer. The types of asphalt cement pavement distresses considered directly related to temperature are 1) low temperature cracking, 2) thermal fatigue cracking and 3) rutting.

Low temperature transverse cracking of asphalt concrete (AC) pavements is a serious problem in the northern United States, Canada and other cold regions. Based on air freezing indices, severe cracking has been reported in areas where the freezing index is equal to or greater than 13,330°C-hours. Low temperature cracking has also been reported in areas where the freezing index is as low as 8665°C-hours (Finn et al. 1976). Figure 1 shows the 8665° and 13,330°C-hours distributions in North America. *Figure 1 shows that flexible pavements in a third of the northern continental United States are affected by low temperature cracking.*

Low temperature cracking discussed in this report does not include cracking due to differential frost heave. This is a serious problem in pavements constructed on frost-susceptible soils. The type of low temperature transverse cracking discussed in this report occurs when the temperature-induced tensile stress in the asphalt concrete (AC) layer in a pavement structure exceeds the tensile strength of the AC layer. Field observations indicate that the cracks start at the surface and progress down through the entire pavement (Burgess et al. 1971). With time and entry of water into the pavement and subgrade through these cracks, the bearing capacity of the pavement system can be reduced, thereby leading to premature failure. During the winter, deicing solutions can enter these cracks and produce localized thawing of the base and/or subgrade. Also, water entering the pavement through these cracks may cause greater differential frost heave, resulting in more cracking.

Another form of cracking related to temperature is thermal fatigue cracking. This form of cracking is most noticeable in the southwestern United States and was first described by Shahin and McCullough (1972). These cracks occur when the asphalt's fatigue resistance is exceeded when it is subjected to repeated moderate daily temperature cycling. The emphasis in this report will be on low temperature cracking.

In the last five years, rutting has become a serious problem in the United States and Canada. There are many reasons for the increased rutting seen today, but the main factor suggested by many pavement engineers is the increase in truck tire pressures. Davis (1987a) found that there are many old pavements in the U.S. today whose bearing capacity could be as low as 345 kPa. This capacity was considered adequate at one time, but not today, as pavements are now being subjected to tire pressures as high as 827 kPa. The use of soft asphalts for minimizing low temperature cracking is considered by many to increase the rutting potential of pavements. This may be true if the mix is incorrectly designed or formulated. Mix design should be based on creep and triaxial compression tests and not on the Marshall design procedure only. Rutting is considered in this report because of the prevailing concern that rutting and the use of soft asphalts go hand in hand.

To reduce the potential of low temperature cracking, soft asphalt cements are currently recommended as binders in cold regions. Based mainly on Canadian studies, the U.S. Army Corps

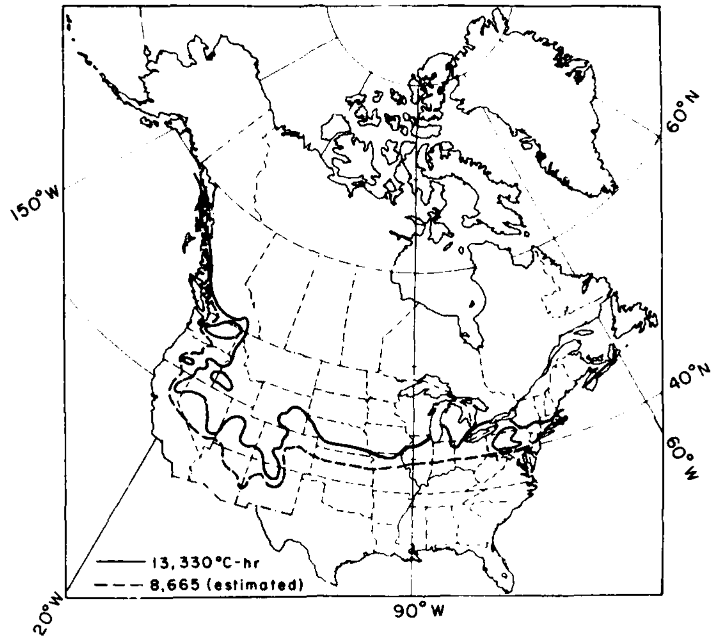

Figure 1. Areas of low temperature cracking in North America.

of Engineers specifies an asphalt whose absolute viscosity at 60°C is 500 poise (Fig. 2) or less in their road and airfield pavements in cold and moderately cold regions. The division between moderately cold and cold is at 1670°C-days. In terms of viscosity grading, these are asphalts characterized in the U.S. as AC 2.5 and AC 5. AC 2.5 asphalts have been used by the Corps of Engineers in Thule, Greenland, and Alaska for military airfields.

Basma and George (1984) also developed a guide map (Fig. 3) for selecting asphalt grades based on temperature data from 175 weather stations. This map is based on limiting rut depths to 12.7 mm and low temperature crack spacing to approximately 115 m/1000 m^2.

A field study was undertaken in the United

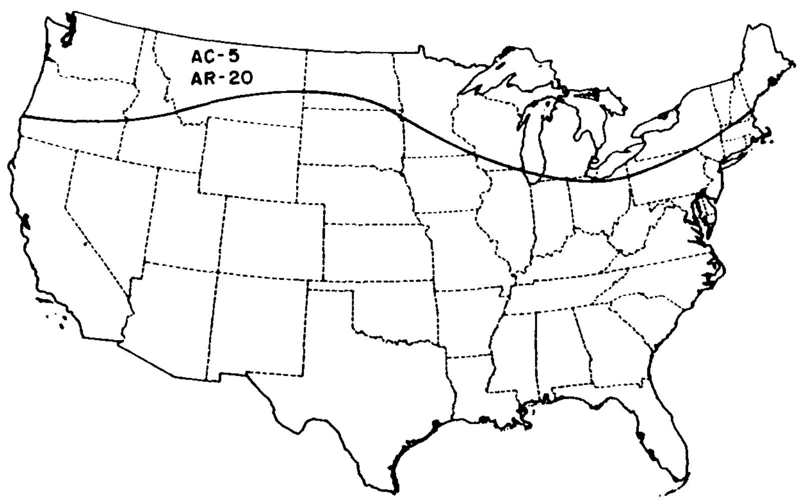

Figure 2. U.S. Army Corps of Engineers recommended asphalt grades for northern Continental U.S.A.

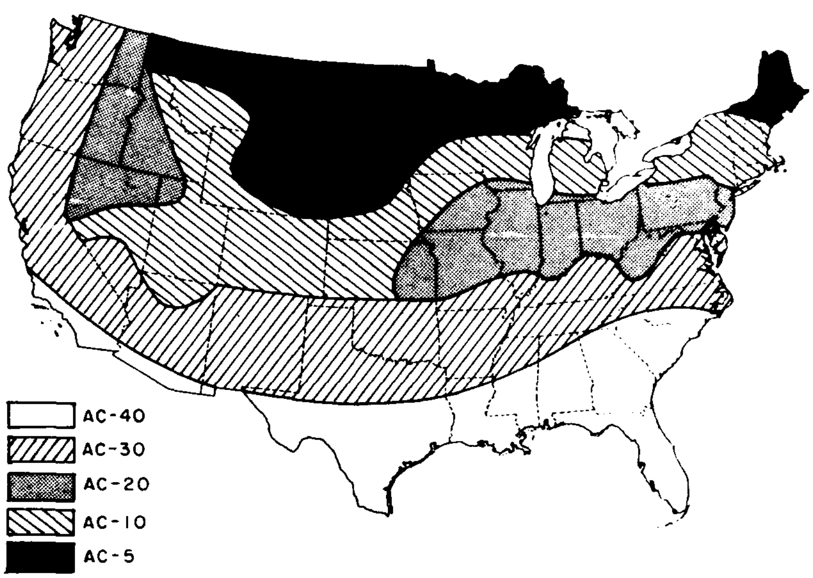

Figure 3. Recommended asphalt grades for Continental U.S. (Basma and George 1984).

States to 1) determine the extent of use of soft grade asphalts by state DOTs and other Federal agencies, and 2) to determine if rutting is a serious problem in states where soft asphalts are used. Twenty-seven states representing the northern USA were contacted, and site visits were made in Maine, Vermont, New York, Michigan, and Alaska. This report presents 1) a brief literature review on low temperature cracking and rutting, 2) a description of the Corps specifications and (3) the results of the field study.

LITERATURE REVIEW ON LOW TEMPERATURE CRACKING

There are several ways to specify the temperature susceptibility of the asphalt; in this report the penetration index (PI), penetration viscosity number (PVN) and viscosity temperature susceptibility (VTS) are used. The Corps of Engineers currently uses the PVN index to specify the temperature susceptibility of the asphalt. A detailed description of PVN can be found in McLeod (1976). It should be stated that like the PI, PVN is an empirical number and is valid only for steam and/or vacuum reduced asphalts. PVN was introduced when it was found that the stiffness of the asphalt calculated using Van der Poel's (1954) nomograph and PI (calculated as the penetration at 25°C and the softening point temperature) produced inaccurate values for waxy asphalts. This was to be expected, since Van der Poel's nomograph was developed for asphalts with a wax content of less than 2%. Based on a study of over 300 asphalts, the PVN can be approximated by

$$\text{PVN} = -11.300 + 1.629 \log(\text{PEN}) + 2.981 \log(\text{VIS})$$

where PEN is the penetration at 25°C and VIS is the kinematic viscosity (cst) at 135°C. The PVN of a given asphalt can also be determined graphically using Figure 4. Most of the asphalts in Canada and the United States are in the PVN range of −1.5 to 0.5 (McLeod 1979).

At present there is major controversy as to whether PI or PVN should be used to characterize the temperature susceptibility of an asphalt binder. The PI currently used for low temperature characterization is determined from penetration measurements at 4°, 10° and 25°C as opposed to that proposed earlier by Pfeiffer and Van Doormal (1936) (penetration at 25°C and softening point temperature). Many feel that since PVN is indexed based on measurements at 25°C and 135°C, it requires a large extrapolation to characterize low temperature behavior. Gaw (1978) compared stiffness calculated using PI and PVN with Van der Poel's nomograph and measured stiffness of different asphalts (high wax and low wax) using the Shell sliding plate rheometer. The comparisons are presented in Figure 5. The data show a better correlation between measured and PI calculated stiffness at low temperatures for high and low wax

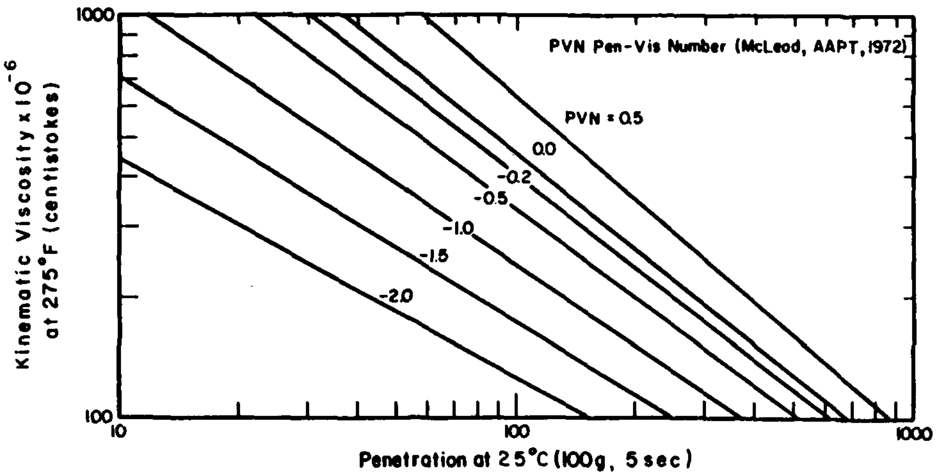

Figure 4. Chart for determining temperature susceptibility (PVN) of asphalt cement.

asphalts. A 2-hour loading time is considered to be appropriate for simulating temperature-induced loading.

Most of our knowledge on low temperature cracking of asphalt pavements comes from test roads in Canada (Burgess et al. 1971, McLeod 1969, 1972, 1978, 1979, Deme and Young 1987). Based on field observations, McLeod (1979), reported on the benefits of using soft asphalts to control low temperature cracking. For example, based on observations on test road 4 in Southern Ontario, he reported 248 type 1 cracks/km in a test section built with 85/100 pen (8.5- to 10-mm penetration) asphalt and 0 cracks in a test section built with 150/200 pen asphalt. (A type 1 crack is transverse across the entire pavement.) Both asphalts were from the same crude source and had the same temperature susceptibility (PVN = –1.35). The aggregate, mixing, construction, etc; were considered constant in both test sections. At the Ste. Anne test road in Manitoba, test sections built with 150/200 pen asphalt over a 2-year period had 162 type 1 cracks/km, whereas the test section constructed with 300/400 pen asphalt had 50 type 1 cracks/km (McLeod 1979). Again, both asphalts had the same temperature susceptibility (PVN = – 1.5). The minimum temperature in Southern Manitoba was –40°C and in Southern Ontario it is seldom below –26°C. These results are tabulated in Table 1.

Based on observations of test roads in Southern

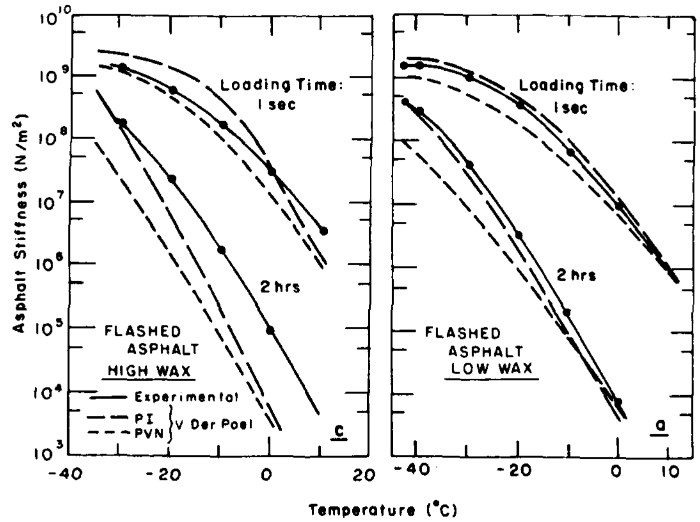

Figure 5. Comparison of calculated and measured asphalt stiffness (Gaw 1978).

Table 1. Asphalt hardness and low temperature cracking.

Test road	*Asphalt*	*Time (yr)*	*PVN*	*Min. temp. (°C)*	*No. of Type 1 cracks/mile*
Test road 4	85/100	4	–1.35	–26	400
Southern Ontario	150/200	4	–1.35	–26	0
Ste. Anne					
Test road,	150/200	2	–1.5	–40	260
Southern Manitoba	300/400	2	–1.5	–40	81

Ontario and the Ste. Anne test road in Southern Manitoba (Table 1), test sections using the same penetration graded asphalt showed the influence of temperature susceptibility on low temperature cracking. For example, at the Ste. Anne test road, test sections constructed with 150/200 pen asphalt with a PVN of –0.6 showed no cracks, whereas the section with a PVN of –1.5 showed 162 cracks/km after 2 years (McLeod 1979).

Deme and Young (1987) reported similar findings based on their field studies at the Ste. Anne test road over a 20-year period. The asphalts used in their study were 1) low viscosity 150/200 penetration grade (high wax content); 2) high viscosity 150/200 penetration grade (low wax content); 3) low viscosity 300/400 penetration grade (high wax content) and 4) high viscosity SC-5. The first three asphalts can be characterized as AC 2.5 with a PVN of –1.5; AC 5 with a PVN of –0.6 and AC 1.5 with a PVN of –1.5, respectively. The average minimum pavement surface temperature during the 1st and 5th winter was –33°C. Unless otherwise stated, the asphalt pavement thickness was 100 mm. Their conclusions are tabulated in Table 2.

The AC 1.5 performed better than the AC 2.5 on a sand subgrade. On the clay subgrade, there were fewer transverse cracks in the pavements with AC 1.5 20-m spacing than in the AC 2.5 8 m. It is also apparent that the 150/200 pen asphalt with a PVN of –0.6 performed better than the 150/200 pen asphalt with a PVN of –1.5. With respect to viscosity grading, an AC 5 asphalt performed better than either the AC 2.5 or AC 1.5. This result clearly shows that using low viscosity (at 60°C) alone will not minimize low temperature cracking. A specification that specifies both viscosity or penetration and temperature susceptibility is required.

Other conclusions reached were that crack spacing varied with subgrade type and pavement thickness. For example, the crack spacing in pavements constructed with AC 2.5 was approximately 2.4 m on sand subgrade, 6.1 m on clay subgrade and 15 m for a 250-mm full depth pavement on clay subgrade. With respect to mix properties, it was found that neither the asphalt content (± 1% of Marshall optimum) nor the filler content (2–5.5% finer than the 75-µm mesh) affect transverse cracking frequency.

As mentioned above, it was found that two asphalt cements with the same penetration at 25°C

Table 2. Some factors influencing low temperature cracking.

Asphalt grade	*PVN*	*Pen*	*Subgrade*	*First crack observed*
AC 1.5	–1.5	300/400	Clay	1st year
			Sand	5th year
AC 2.5	–1.5	150/200	Clay	1st year
			Sand	1st year
			250-mm full depth on clay	1st year
AC 5.0	–0.6	150/200	Clay	9th year
			Sand	5th year

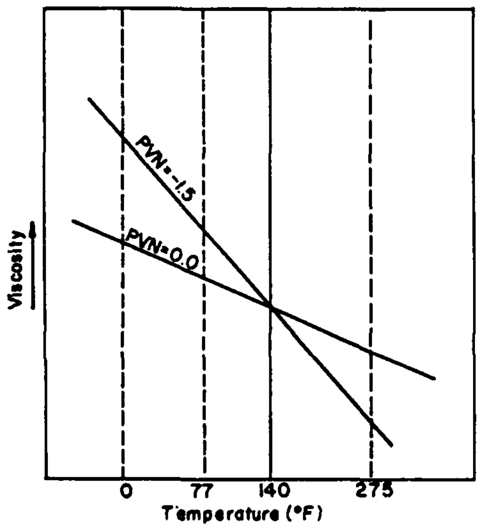

Figure 6. Influence of temperature susceptibility on asphalt stiffness.

or viscosities at 60°C did not behave similarly at other temperatures. The rate at which the viscosity or penetration changes with temperature (see Fig. 6) is characterized as the temperature susceptibility of the asphalt and has been found to be dependent on the crude source. The temperature susceptibility of an asphalt gives an indication of the rate of change of stiffness with temperature; i.e., the slope of the viscosity vs temperature curve of a low-temperature-susceptible asphalt is less than that of a high-temperature-susceptible asphalt, as illustrated in Figure 4.

Some of the results of the Ontario and Ste. Anne test roads, have been substantiated in the laboratory. For example, Sugawara et al. (1982) concluded from their studies on asphalt concrete beams in Japan that the fracture temperature decreased in beams constructed with soft asphalts (increased penetration values) as shown in Figure 7. With the exception of asphalt 1, the induced thermal stress at fracture was 30 kg/cm^2 and can be considered to be independent of the asphalt grade. The thermal stresses induced in the asphalt concrete were also independent of the temperature susceptibility of the asphalt as shown in Figure 8. Figure 8 also shows that the fracture temperature was dependent on the temperature susceptibility of the asphalt.

Sugawara and Moriyoshi (1984) studied the influence of mix properties such as air voids, aggregate gradation and asphalt content on fracture stress at low temperatures on AC beams cooled at –30°C/hr. Previously, Sugawara et al. (1982) found that the cooling rate (5°C/hr to 30°C/hr) had no influence on fracture strength or fracture temperature. Similar results were reported by Fabb (1974). As shown in Figure 9, the air void content in a dense-graded mix (asphalt content = 5.8%), had an influence on the fracture stress but not on the fracture temperature.

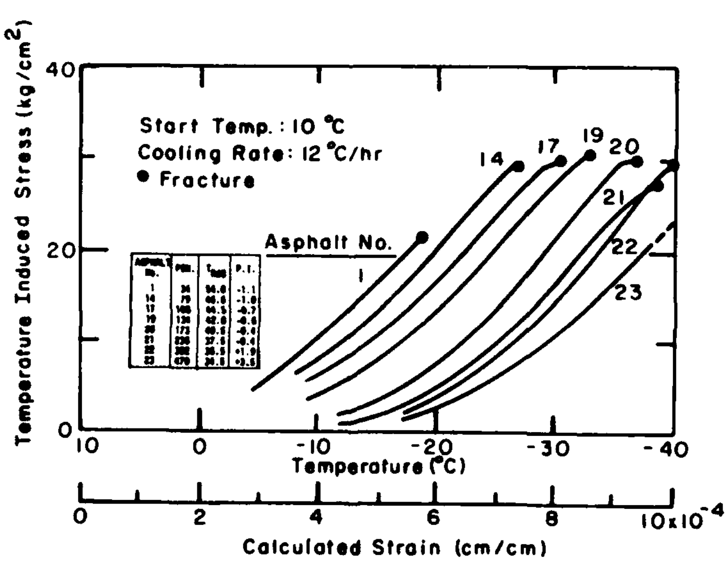

Figure 7. Influence of penetration on temperature-induced stress and fracture (Sugawara 1982).

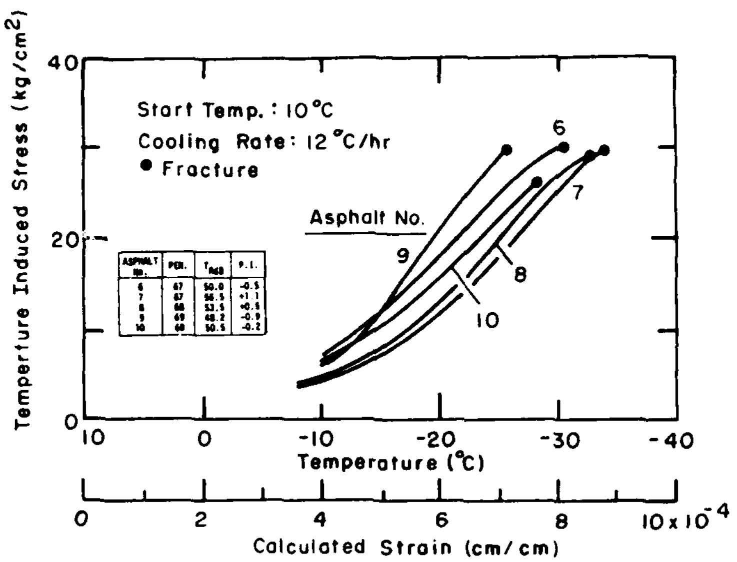

Figure 8. Influence of temperature susceptibility on temperature-induced stress and fracture (Sugawara 1982).

The influence of aggregate gradation on thermal stresses was studied using four types of mixes: 1) dense-graded (type A), 2) fine dense-graded (type B), 3) coarse-graded (type C), and 4) stabilized asphalt base material (type D) (Sugawara and Moriyoshi 1984). These gradations are shown in Figures 10 and 11. For comparison purpose, the New Hampshire DPW highway type E wearing course (100% passing the 1.3-cm sieve), and type B base course gradation are shown in Figures 10 and 11, respectively. The dense-graded type A mix is very similar to the N.H. type E mix and the type C coarse-graded mix is very similar to the type B N.H. base course mix. As shown in Figure 12, there is a difference between the temperature-induced fracture strengths of the coarse and dense-graded mix. For example, the difference in strength between type A and C mix is 12 kgf/cm². A larger

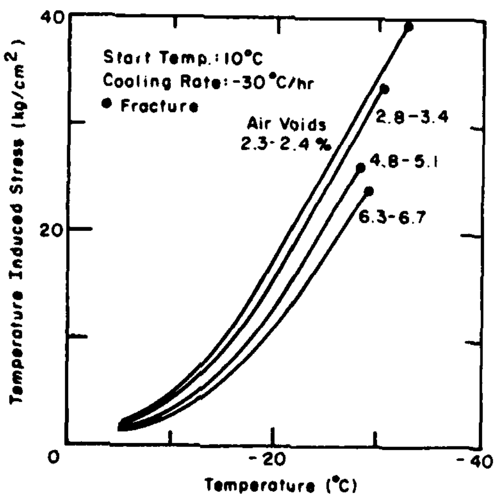

Figure 9. Influence of air void content on thermal fracture (Sugawara and Moriyoshi 1984).

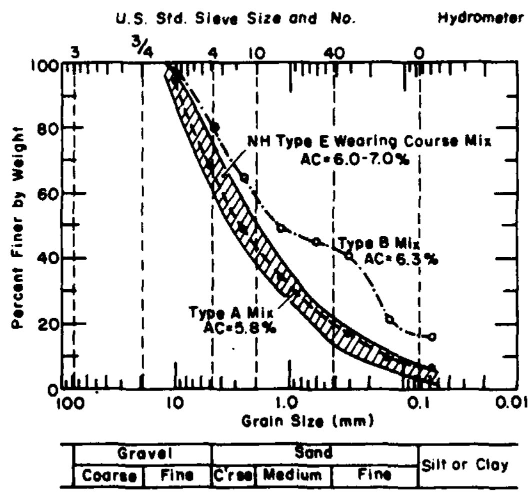

Figure 10. Dense-graded (type A) and fine dense-graded (type B) aggregate gradation.

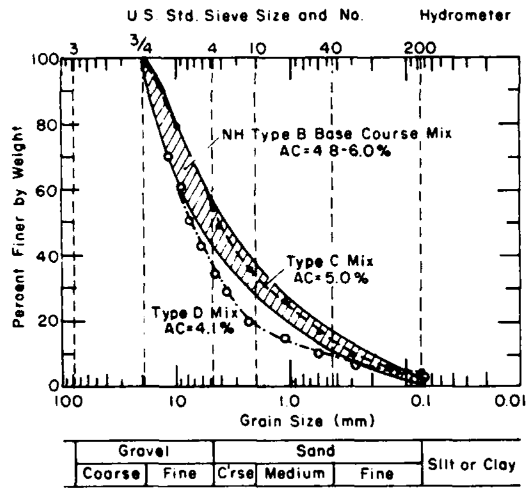

Figure 11. Coarse-graded (type C) and stabilized asphalt base (type D) aggregate gradation.

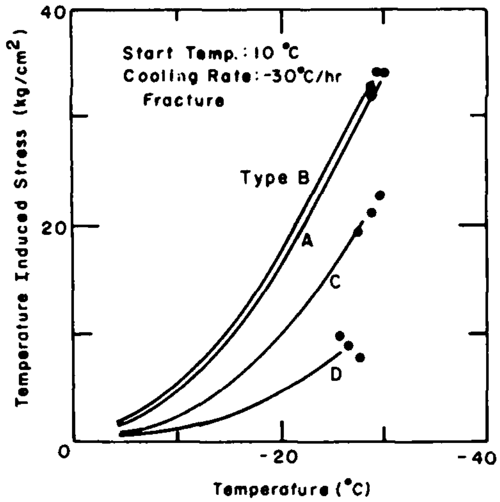

Figure 12. Effect of mixture type on thermal fracture (Sugawara and Moriyoshi 1984).

difference is seen between the stabilized mix (type D) and the dense-graded mix. However, the fracture temperatures are similar to one another (approximately –30°C). Another observation that can be made from Figure 12 is that increasing the fine content does not influence the temperature-induced strength of the mix. The influence of gradation will become an important issue with respect to low temperature cracking as there is a trend toward using a coarser mix for minimizing rutting.

The above results (Fig. 7 and 8) suggest that the asphalt binder controls the temperature at which fracture occurs. Fabb (1974) also concluded the same from his study on the influence of asphalt mixture on low temperature cracking. The failure stress (Fig. 9 and 12), however, appears to be dominated by the stiffness modulus ($S_{t,T}$) of the mixture as evidenced by the decreasing stress with increasing air voids and changing aggregate gradation. This is to be expected as the thermal stress in a material is a function of not only temperature but also of the coefficient of thermal contraction and the stiffness modulus of the mixture. The fracture temperature, tells us when the asphalt cracks (fails). It does not tell us the distance or time

Table 3. Influence of asphalt and mixture PPTs and environment on low-temperature cracking.

	Fracture temperature	*Temperature-induced fracture stress*	*Frequency of cracking (cracks/mile)*
Asphalt PPTS			
Asphalt grade	Yes	No	Yes
Temp. susceptibility	Yes	No	Yes
Pavement thickness	No	?	Yes
Mixture PPTS			
Filler content	No	No	No
Air void	No	Yes	No
Asphalt content ±1% of optimum	No	Yes	No
Gradation	No	Yes	No
Environment			
Rate of cooling	No	No	No
Subgrade type	No	No	Yes

it takes for the crack to propagate through the mixture, as the structure will retard/control further crack propagation.

In summary (see Table 3), based on the limited field and laboratory data, a combination of asphalt grade and temperature susceptibility influences the temperature at which fracture occurs and the frequency (cracks/mile) of fracture. The fracture temperature was found to be independent of the cooling rate (≥ 5°C/hour.) The temperature-induced stress in a mixture is independent of the asphalt grade or temperature susceptibility. It is dependent on the air voids content aggregate gradation, asphalt content and fine content. The frequency of cracking was also found to be dependent on the subgrade type and pavement thickness.

RUTTING

In the last five years, permanent deformation (commonly known as rutting) of flexible pavements has become a serious problem in the United States and Canada. Rutting is usually seen as channelized wheel paths in airfield and highway pavements and in highway intersections. It can be a safety hazard when water in the ruts turns to ice or when trapped water in the ruts creates a condition for hydroplaning. Rutting in the outer wheel paths can also lead to longitudinal cracks in the shoulders. Lister (1972) reported that when rut depths reached 10 mm, cracking was seen in the pavements. These cracks may allow influx of surface water, which can lead to a reduction in the base, subbase and/or subgrade strength. With the passage of time and traffic the life of the pavement can be significantly reduced.

The author was unable to locate a definition of rutting in terms of engineering parameters, such as strength, stress-strain, modulus, etc. Morris and Haas (1972) attempted to define rutting by describing the phenomenon as follows:

> Rutting of asphalt pavements manifests itself in surface longitudinal depressions in the wheel paths caused by lateral distortion and/or consolidation in one or more of the component pavement layers due to repeated transient load applications.

What are the factors that control rutting? Any of the mix design components individually or in combination, such as *asphalt type, asphalt content, aggregate type, aggregate gradation, air void content, stability, flow,* etc., are considered to have a major influence on rutting. Other factors such as *poor subgrade preparation or compaction, inadequate base or subbase compaction, larger traffic volume, higher tire pressures and studded tires* have also been cited. It is interesting to note that when rutting is observed, the first change in design is usually the grade of asphalt, typically to a harder grade. This change will probably help to a certain extent, but it is not guaranteed to solve the problem as rutting has been observed in pavements constructed with harder asphalts. Recently, several studies have been initiated in this country and Canada to understand the problem of rutting or permanent deformation. A summary of results from laboratory and field studies conducted in previous years on rutting is presented below.

Data obtained from the AASHTO Road Test in Ottawa, Illinois, provided the first insight on rutting. Finn (1963) concluded from the AASHTO results that subgrade rutting could be reduced by increasing the pavement thickness and increasing the density requirements of each layer of the pavement structure. With respect to the asphalt concrete layer, Vesic and Domaschuk (1964) concluded from the AASHTO Road Test results that approximately 70% of the rut was due to lateral displacement and the rest was due to densification of the pavement. Others (Goetz et al. 1957, Palmer and Thomas 1968, and Epps 1970) also concluded that densification took place throughout the width of the pavement and rutting was due primarily to lateral distortion. Finally, it was also concluded from the AASHTO Road Test and other studies (Seed et al. 1962, Lister 1972) that rutting occurred throughout the pavement structure (surface and base course).

From the above studies, it is apparent that rutting occurs when any part of the pavement structure is unstable under loading. Assuming the subgrade was compacted to the required density, then we can safely assume that any rutting seen will be due to an unstable asphalt concrete mix. The two components of a mix are the aggregates and the asphalt cement. What is the influence of these components on rutting?

Hofstra and Klomp (1972), in the Netherlands, conducted a laboratory study on the influence of the asphalt mixture components on rutting. The study was in response to the European problem with rutting in their flexible pavements in the early seventies. The experiments were conducted under well-controlled temperatures and traffic was simulated using a 0.7-m-wide × 3.25-m outside diameter circular test track. The wheel loads

Table 4. Composition and Marshall properties of asphalt mixes included in investigation (Hofstra and Klomp 1972).

			Asphalt mixes applied					
			Asphaltic concrete			*Sand asphalt*		
			I	*II*	*III*	*I*	*II*	*III*
Composition:								
porphyrl (2–12 mm)		%w	55	55	55	—	—	—
sand		%w	35	35	35	82.5	82.5	—
sand (crushed)		%w	—	—	—	—	—	82.5
filler		%w	10	10	10	17.5	17.5	17.5
bitumen	50/60 pen	pha	6	—	—	—	—	
bitumen	40/50 pen	pha	—	—	—	7	9	7
bitumen	80/100 pen	pha	—	6	—	—	—	
bitumen	100/29 pen	pha	—	—	6	—	—	—
Marshall properties:								
Stability (N)	—		7500	5800	5300	8850	*	15,500
Flow (mm)		—	5.5	6.1	6.0	4.4	*	3.2
Void content (90 V)		—	1.7	1.8	1.7	3.6	4.9	8.0

and speeds could be varied from 330 to 1500 to 20,000 N and from creep to 16 km/hr, respectively. The temperature of the asphaltic layers could be regulated from 20° to 60°C.

The test track consisted of six or seven, 1.2-m-long asphalt concrete test sections. The mix was compacted in 5-cm lifts to 99–100% Marshall density on top of a stable sand subgrade. The variables in the test sections were thickness of asphaltic layer and the properties of the mix, basically asphalt grade. The different asphalt mixes used are presented in Table 4. In most test sections, a 10-cm minimum thickness was chosen so that rutting was due to plastic deformation of the asphalt mix only. Higher than optimum asphalt content and natural sand were used in the mixes so as to increase its rutting potential. All the test sections were subjected to 483 kPa contact stress at a wheel speed of 15.3 km/hr.

Hofstra and Klomp (1972) concluded from their study that 1) deformation was due to transverse movement of material as density measurements taken in the wheel path and outside the wheel path were similar; 2) use of a harder grade asphalt (50/60 pen bitumen) produced less rutting; 3) the rut depth increased with increasing number of load repetitions; and 4) however, when round sand is replaced with crushed sand and bitumen content decreased, there was a major improvement in rut resistance. The asphalt used was a 40/50 pen grade. No tests were conducted with crushed sand and 180/200 pen grade asphalt.

France also had a major problem with rutting at about the same time as in the Netherlands (Uge and Van De Loo 1974).The French felt that the major reason for rutting was the heavy trucks on their roads (the maximum allowable load for a single axle truck was 26 kips [116 kN]). Other factors contributing to the rutting were inadequate structural design, poor material quality and insufficient compaction.

Several test sections were built, where the design was modified by using more angular aggregates, higher percentage of stones and fines and less asphalt content. It is assumed that the regular mix design was similar to the Dutch (i.e., predominantly rounded aggregates). One field experiment, using the modified mix design, was conducted on Route Nationale 113 near La Fare les Oliviers, northwest of Marseille (Uge and Van De Loo 1974). Two 700-m length test sections were constructed on a 5% slope. The average daily traffic (ADT) on this route was 15,000 vehicles of which 22% were heavy trucks. The pavement surface temperatures reached 60°C.

The mix contained very angular aggregates, a high percentage of fines (9% in the wearing course, 7% in the base course and 3% in the road base passing the no. 200 sieve), low binder content, and hard asphalt (40/50 pen). A 400-m test section was also constructed using a 40/50 pen grade, low-temperature-susceptible asphalt (PI = 2). This asphalt was designated 40/50S. Compaction was carried out with very heavy pneumatic-tired rollers on thick layers. The percentage of air void was high after compaction and was reported unchanged

after traffic application. Rutting was not seen on this test section after four years of service but thermal cracks were noticed in both the 40/50 and 40/50S sections.

Other observations made were that the rut resistant mix was prone to mixing problems, there was segregation of coarse aggregates, and the mix had a high air void content (10%). Because of this high air void content and low asphalt content, the pavements aged faster, becoming brittle and less durable. However, it was adopted by the French because the modified mix seemed to overcome the rutting problem.

Uge and Van De Loo (1974) studied the influence of mix design parameters on mix stability. They reported a poor correlation between Marshall test results and rutting. They reported similar observations made by Hofstra and Klomp (1972) that more rutting was found in sandsheet asphalt (i.e., a mixture of rounded sand and asphalt, usually confined to the surface course) than in gravel sand asphalt mixes, as shown in Figure 13. Other observations made were 1) continuous-graded asphaltic concrete mixes were more stable than gap graded mixes; and 2) as expected, angular crushed aggregates were more rut resistant. They looked at the influence of using only crushed sand with rounded coarse aggregates in the mix, instead of the usual practice of using crushed coarse aggregates and natural sand, and found that they got better performance with the crushed sand. Also using rounded coarse aggregate will probably increase the workability of the mix and reduce the cost of the mix with respect to 100% crushed requirements. This influence of the shape of the aggregates on mix stability is shown in Figure 14. On the question of the type of binder, they recommended a hard bitumen with low temperature susceptibility (PI = +2).

In the early eighties, a series of workshops by AASHTO was conducted in the western United States to address the severe problem of rutting in their highways. Some of these states began to have this problem in the mid to late seventies and the severity ranged from 13- to 38-mm rut depths. The states began to see rutting in pavements that had performed well for over 10 years. They considered the change in traffic loading, (i.e., increased tire pressure, tire configuration, increased gross loading and load repetition) as a major contributor to rutting. This change in loading exceeded the bearing capacity of pavements and could explain why previously well performing pavements are now failing. Another contributing factor was thought to be the soft asphalts used for reducing low temperature cracking.

Some of the recommendations that the study (Betenson et al. 1984) made with respect to rutting to the western states were as follows:

1. Require that a minimum of 60% of the material retained on the no. 4 sieve have two fractured faces.

2. Use the no. 4, 40 and 200 sieves as control sieves for the fine fraction of the mix. For near

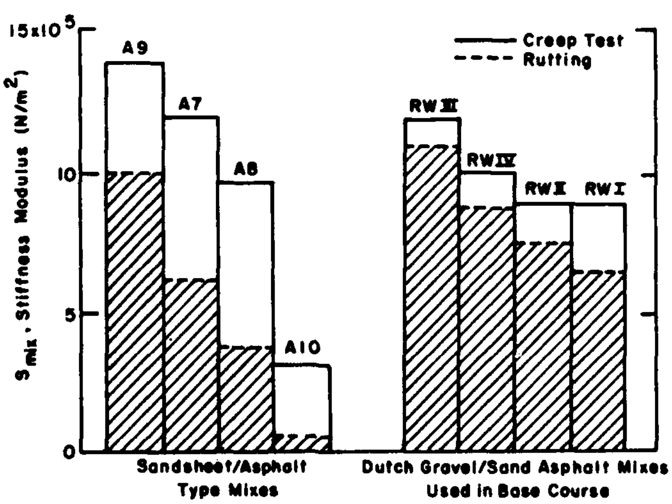

Figure 13. Influence of aggregate shape and size on rutting potential of asphalt concrete mixtures (Uge and Van de Loo 1974).

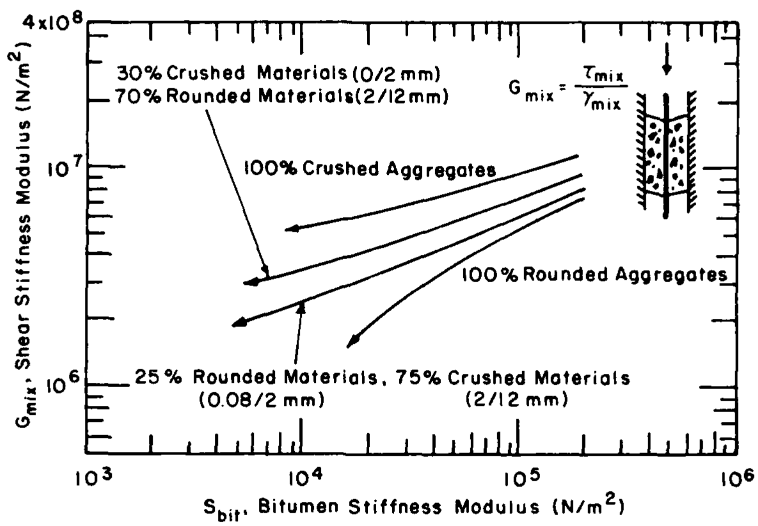

Figure 14. Influence of crushed material on rutting potential of asphalt concrete mixtures (Uge and Van de Loo 1974).

maximum density, it was recommended that it not exceed the following gradation:

Sieve no.	*Percent passing (%)*
4	55
10	37
40	16
200	3–7

if the percent passing any of these control sieves is exceeded, the mix is considered to have an increased potential for rutting.

3. Control the field compaction by specifying a percentage of the measured zero air void mix determined by the Rice method (ASTM D2041). The study recommended the acceptable density to be more than 94% of the measured zero air void mix or not less than 91% based on the mean of five tests.

Brown (1987) suggested that specifying the density based on the Rice method can lead to over-asphalting the mix, as the mix can be made voidless by adding more asphalt (and therefore meeting density requirements) which will then lead to flushing and rutting of the pavement. Brown (1987) recommended that field density specifications continue to be based on a percentage of Marshall laboratory density. States that use this method counter that asphalt content of the mix is monitored periodically and therefore they will know if the mix was over-asphalted to meet density requirements.

4. No recommendations were made to the states to change the asphalt to a harder grade. However, the states were willing to accept some cracking if using harder asphalts would reduce rutting. A cooperative study was proposed to compare asphalts from rutted and nonrutted pavements. This study has been deferred to the Strategic Highway Research Program (SHRP).

Wyoming, one of the participants of the above workshop, conducted an informal study on rutting (Wyoming State Highway Department 1982). The study involved 11 rutted and 7 non-rutted roads. Based on a statistical analysis (some of the variables involved were design, construction procedure and quality, asphalt content, surface densities, aggregate properties and asphalt properties), with respect to asphalt properties, the conclusion reached was that cement "hardness" had a significant influence on rutting. The study found that asphalts with low penetration and high viscosity performed better. The Wyoming study also concluded that traffic, aggregate shape and poor compaction were other factors that produced rutted pavements. However, based on the finding on the influence of asphalt grade, Wyoming recommended a change from AC 10 to AC 20, which is now used.

Table 5. Testing program for study sites (Huber 1987).

Property	*Design*	*Construction*	*Present*
Percent asphalt	X	X	X
Aggregate gradation	X	X	X
Crushed aggregate faces	X	X	X
Percent air voids	X	X	X
Percent VMA	X	X	X
Marshall stability	X	X	X
Hveem stability		X	X
Asphalt penetration (25°C)			X
Asphalt viscosity (60°C)			X
Aggregate BSG	X	X*	X*
Rice TSG	X	X*	X*
Rut depth		X†	X
Layer thickness	X	X	X

* Assumed equal to design.
† Assumed equal to zero.

A more recent field and laboratory study was conducted to determine why some pavements in Saskatchewan, Canada, rutted and others did not. Huber (1987) identified highway sections that carried similar traffic volume but exhibited different performance with respect to rutting. The laboratory study involved a testing program on core samples taken in the outer wheel path and between the wheel paths. The cores from the outer wheel path were considered to represent the present condition of the pavement subjected to traffic loading (present), whereas the cores from between the wheel path were supposed to represent the pavement at the end of construction prior to any traffic loading (post construction). The testing program is shown in Table 5. The description of the sites is given in Table 6. Note that soft asphalts (AC 1.5, AC 5 and AC 6) were used and that summer pavement temperatures in Saskatchewan can be as high as 60°C.

A statistical analysis was conducted on the mix property results obtained from the laboratory test program. A poor correlation was found between rut depth and the mix parameters for both the post-construction and present condition data, as shown in Table 7. Huber (1987) suggested that there is a correlation between rutting rate (millimeters per million equivalent single axle loads) and asphalt content, Hveem stability, air voids and voids filled, based on post-construction data. The relationships for rutting rate and asphalt content and voids filled are shown in Figures 15

Table 6. Sites in Saskatchewan assessed for rutting behavior (Huber 1987).

Site no.	*Location*	*Structure type*	*Traffic speed*	*Asphalt cement*	*Year of construction*	*Age % life*	*Rutting performance*
1	C.S. 1–8 EB km 4.24	Overlay	Fast	AC 1.5	1977	55	Poor
2	C.S. 1–8 EB km 1.44	Overlay	Fast	AC 1.5	1977	55	Fair
3	C.S. 1–19 MB km 4.6	Conv.	Fast	AC 1.5	1983	40	Poor
4	C.S. 1–19 WB km 7.0	Conv.	Fast	AC 1.5	1983	40	Poor
5	C.S. 1–19 WB km 9.0	Conv.	Fast	AC 6	1983	40	Fair
6	C.S. 1–19 EB km 19.5	Overlay	Fast	AC 6	1984	5	Good
7	C.S. 9–5 SB km 17.1	Overlay/ Recycle	Slow	AC 1.5	1982	15	Poor
8	C.S. 9–5 SB km 19.5	Overlay	Fast	AC 6	1981	25	Fair
9	C.S. 9–6 SB km 16.1	FDAC*	Fast	AC 5	1972	90	Good
10	C.S. 22–2 WB km 24.75	Overlay/	Slow	AC 1.5	1982	20	Good
11	C.S. 80–1 WB km 10.13	FDAC	Slow	AC 6	1975	200	Good

*Full depth asphalt concrete.

Table 7. Summary of regression analysis using average parameter data (Huber 1987).

	Post-construction				Present condition			
	Rut depth (mm)		Rutting rate ($mm/10^6$ ESAL)		Rut depth (mm)		Rutting rate ($mm/10^6$ ESAL)	
Parameter	Correlation (r^2)	Probability	Correlation (r^2)	Probability	Correlation (r^2)	Probability	Correlation (r^2)	Probability
Accumulated ESAL's	0.099	0.410*	—	—	0.099	0.410	—	—
AC content	0.134	0.332	0.673	0.007	0.082	0.456	0.667	0.007
Fracture	0.269	0.153	0.229	0.193	0.074	0.516	0.004	0.881
Air voids	0.221	0.202	0.483	0.038	0.182	0.252	0.234	0.187
Voids in the mineral aggregate	0.111	0.379	0.124	0.352	0.001	0.927	0.262	0.159
Hveem stability	0.211	0.213	0.490	0.037	0.279	0.144	0.489	0.036
Marshall stability	0.110	0.380	0.056	0.540	0.019	0.723	0.227	0.194
Penetration	—	—	—	—	0.004	0.879	0.187	0.245
Viscosity	—	—	—	—	0.042	0.599	0.082	0.456
Voids filled	—	—	0.530	0.026	0.211	0.213	0.282	0.141
Flow	—	—	0.027	0.301	—	—	—	—

and 16. The author believes that, at best, there is only a weak correlation between the rutting rate and the variables shown in Figures 15 and 16.

Based on the poor correlation Huber (1987) conducted a threshold analysis on air voids, VMA, asphalt content, voids filled, fractured faces, Marshall stability and Hveem stability using the post-construction data. Acceptable rutting was defined as 20 mm at the end of the pavement design life. The projected rut depths at the end of the design life of the pavement were calculated by linear extrapolation of the current rut depth with respect to remaining years of the design life. The

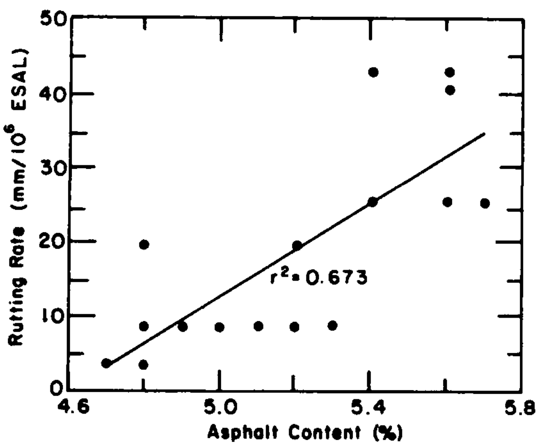

Figure 15. Correlation between asphalt content and rutting rate (Huber and Herman 1978).

rut depth prediction is presented in Table 8. In this way, Huber was able to identify acceptable and unacceptable sites based on current rut depths.

Huber's (1987) conclusions from the threshold analysis are summarized in Table 9. Basically, he found (based on post-construction test data) that at the time of construction 1) *mixtures with an air void content of 4% and above* had less rutting (Bissada [1983] and Ford [1985]) reported similar observations), 2) mixtures *with VMA of 13.5% or higher* performed better (this is similar to results reported by Ford [1985]), and 3) that mixtures constructed *with an asphalt conte.it of 5% or less performed better.*

The above results suggest that a leaner mix is less prone to rutting and that the leanness of a mix could be characterized by the Voids Filled Index. The influence of voids filled on rutting can be seen in Figure 17. There is a definite threshold level between voids filled and performance. The results indicate that mixtures with 60% to 65% voids filled performed better than those with the higher percentages recommended by the Asphalt Institute (75%–85%). Also, Huber (1987) concluded that his results did not show a clear division between acceptable and unacceptable sites with respect to fractured faces and Marshall and Hveem stability values. With respect to fractured faces, Huber (1987) commented that a combination of factors such as high asphalt content and high voids filled could have masked the effect of the high fractured-faced aggregates.

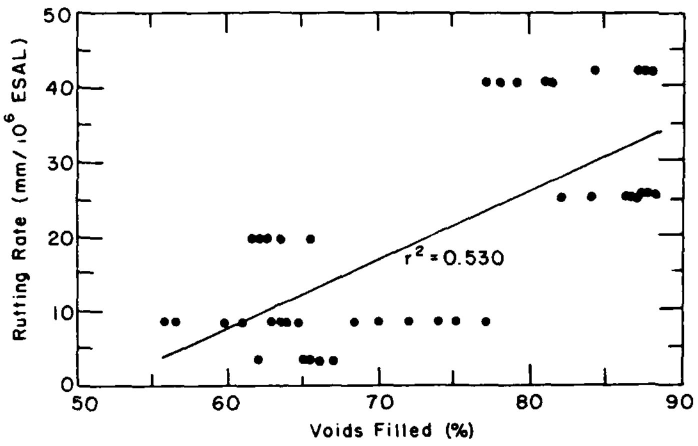

Figure 16. Correlation between voids filled and rutting rate (Huber and Herman 1978).

Huber (1987) also noted that the acceptable sites had a lower asphalt content and higher voids (i.e., a leaner) mix than that specified by the mix design. The unacceptable sites were found to meet the design asphalt content requirements and lower air void (i.e., a tighter mix). Huber (1987) also concluded that the asphalt type did not have a significant effect on rutting performance. It is evident from this review on rutting that *the overall performance of the mixture and not of the asphalt cement alone is the major factor for controlling rutting.*

Results cited from the studies above indicate that the required mixture properties for a rut resistant pavement are not necessarily contrary to those necessary for a thermal crack resistant pavement. A summary of the influence of the asphalt and mixture properties on rutting and low temperature cracking is presented in Table 10. The studies mentioned above strongly recommend using crushed aggregates, leaner mixtures to reduce rutting and softer grade, low- temperature-susceptible asphalts for minimizing low temperature cracking.

One commonly cited factor influencing rutting is high tire pressure. AASHTO and FHWA held a symposium on the influence of high truck tire pressure on pavement systems (AASHTO and FHWA 1987). The states perceive that the change from bias ply to radial tires (cold inflation tire pressure rating of 105 to 120 psi) produced a significant increase in pavement rutting. The use of radial tires allowed trucks to haul heavier loads.

Another point related to the high tire pressures is that the design of pavements (thickness) is based on results from the AASHTO test road where the maximum axle load was 18 kips and tire pressures

Table 8. Future rut depth prediction (Huber 1987).

Site no.	*Rut depth (mm)*	*Age (yr)*	*Percent life used*	*Predicted rut depth (mm)*	*Acceptability*
1	15.0	7.7	51	29.3	No
2	6.5	7.7	51	12.7	Yes
3	10.0	1.4	28	35.3	No
4	6.0	1.4	28	21.1	No
5	4.5	1.4	28	15.9	Yes
7	15.0	2.8	18	81.8	No
8	12.0	3.8	25	48.0	No
9	11.5	14.8	98	11.7	Yes
10	3.0	2.8	18	16.4	Yes

Table 9. Threshold analysis results (Huber 1987).

Parameter	*Threshold value*
Air voids	4% minimum
Voids in the mineral aggregate	13.5% minimum
Asphalt content	5.1% maximum
Voids filled	70% maximum
Fractured faces	60% minimum
Marshall stability	—
Hveem stability	37% minimum

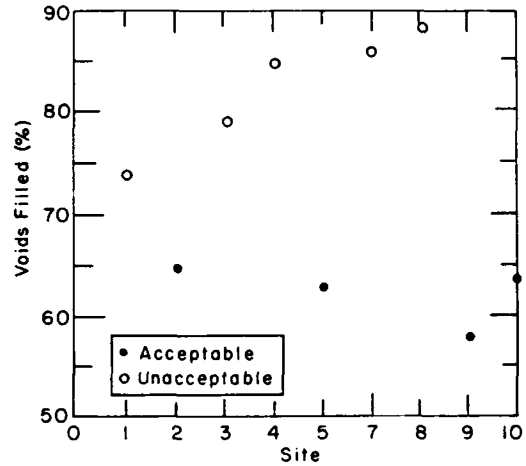

Figure 17. Threshold analysis on voids filled (Huber and Herman 1978).

between 517 and 552 kPa. At present, the load limit has been increased to 89 kN for a single axle and 151 kN for tandem axles and truck tire pressures have been measured as high as 827 kPa. At a minimum, the present empirical pavement thickness design method must be upgraded to account for the change in loading characteristics. The upgrade can be seen as the trend towards a "mechanistic" design of pavements, based on multilayered linear elastic theory. The theory will require a reliable input for the layer properties, which means for the asphalt concrete layers it will require mixture properties and not the asphalt properties alone.

Rutting of flexible pavements from increased tire pressures can be of two types: 1) load-induced deformation (bearing capacity failure) and 2) slip deformation (shear failure). Excessive rutting is seen in the hot summer months because of the dependence on the asphalt binder to provide the shear strength (cohesive) and not on the interlocking of the aggregates (frictional). On open highways, rutting is mainly due to repeated application of loads and deformation is of the load induced type. This load is applied to the pavement by the tire through its sidewall and inflation pressure. This type of deformation occurs under all tires (i.e., under the free rolling and driving tires). At intersections, rutting consists of both load induced and slip deformation. Slip deformation occurs under tires that are being driven or braked. The rut depth at intersections is a function of the mass of the vehicle and momentum applied by the vehicle during acceleration or deceleration at the intersection and the relative resistance of the tire and surface material. The braking force applied at

Table 10. Influence of asphalt and mixture properties on low-temperature cracking and rutting.

Asphalt and mixture properties	*Low temperature cracking*	*Rutting*
1. Asphalt grade	Yes (major)	Yes (minor)
2. Asphalt temperature susceptibility	Yes (major)	Yes (minor)
3. Asphalt content (±1% of optimum)	No	Yes
4. Aggregate shape	No	Yes
5. Air void content	No	Yes
6. Aggregate gradation	No	Yes
7. Large traffic volume	No	Yes
8. High tire pressure	No	Yes
9. Studded tires	No	Yes

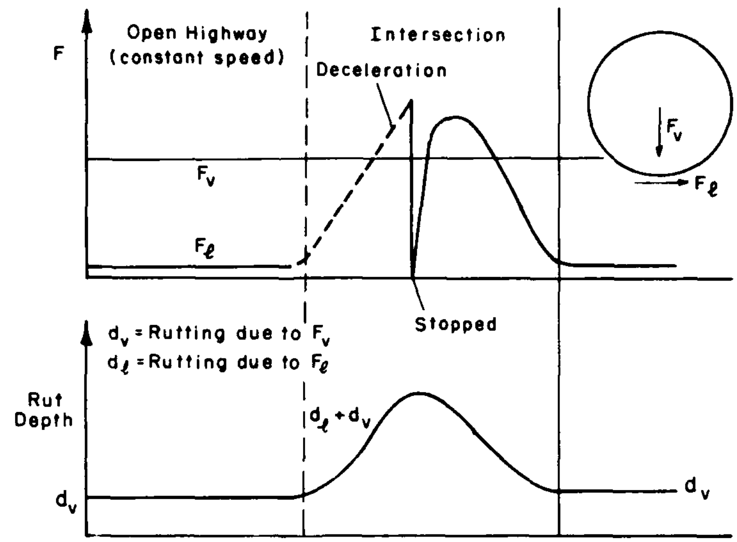

Figure 18. How load application affects rut depth.

intersections, result in the often seen shoving of pavements. As the vehicle stands at the intersection, the pavement is also subjected to a long term (relative to open highway) vertical load. This increased time of loading produces the deep vertical ruts seen at intersections. As the vehicle accelerates away from the intersection, the shear forces applied through the driving tires can "eat away" pieces of the pavement. This form of rutting is probably more common in states where studded tires are still allowed. The different forces and ruts are illustrated in Figure 18.

The loading conditions at intersections are different than that on the open highway. Therefore, *the structural mix design for intersections should be different from that of an open highway.* Laboratory

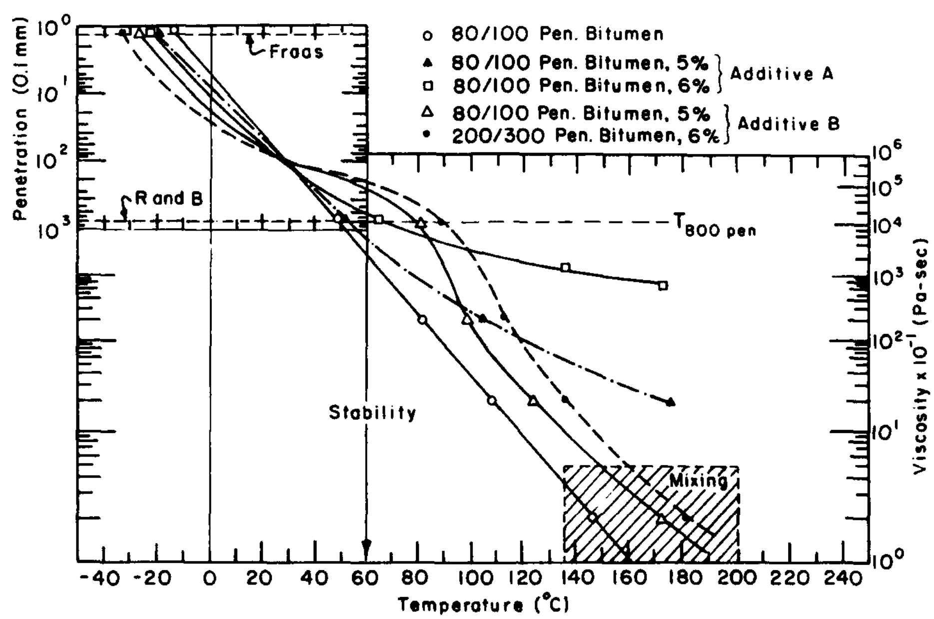

Figure 19. Influence of different additives on asphalt cement properties (Deme 1988).

tests for characterizing the mix should include compressive creep test, some form of shear test, such as the direct shear test and triaxial test at elevated temperatures.

Additives to the asphalt cement are currently being studied for minimizing rutting in asphalt concrete pavements. The additives basically change the properties of the asphalt at high temperatures, usually by increasing the viscosity, as shown in Figure 19. Overall, this will increase the cohesive strength of the asphalt concrete mixture. Again, this only maximizes a small component of the asphalt aggregate mixture and in many cases it is a costly alternative. It is more appropriate to use additives for controlling low temperature cracking, since a major component in low temperature cracking is the tensile strength of the asphalt cement.

Finally, rutting can be controlled by using larger stones in the mix. Davis (1987b), found pavements constructed with soft asphalts and large 7.6-cm top size stones performing well after 50 years of service. These pavements were compacted to less than 2% air voids. Acott (1987) presented an example of roads in Tennessee (15.7 km, on I-65 between state road 11 and state road 50) constructed from large stones and 150/200 pen asphalt. The road had 30.5-cm black open-graded base course of which the bottom 17.8 cm consisted of 7.6 cm top size stones. A 2.5-cm surface course was used. The road was constructed in 1967 and was overlaid in 1983 and 1987. The reason for the overlay was because of shrinkage cracking on the surface. Rutting was not a problem on this section. Other observations reported were that 1) the permeability of the pavement structure was very high; 2) after 20 years, the asphalt had aged from a 150 pen to a 75 pen; 3) the asphalt film thickness in the base course was approximately 10 times thicker than in a conventional dense mix. A similar account on a road section in Indiana was presented. The use of large stones is not new (Warren 1903) but it looks very promising for controlling rutting and should be pursued further.

U.S. ARMY CORPS OF ENGINEERS ASPHALT SPECIFICATION

Specifications for selecting asphalts are currently undergoing revision. However, with respect to cold regions, the basics still remain from the 1976 specifications. Based on the test road studies conducted by McLeod (1969, 1972, 1976, 1978, 1979) in Canada, the U.S. Army Corps of Engineers currently specifies soft asphalts in Army installations in the Northern United States and uses the PVN method for specifying temperature susceptibility. It should be noted that in the U.S., only the Corps of Engineers has a temperature susceptibility specification on its asphalt. The Corps of Engineers specifies asphalts with a minimum PVN of –0.5 and lower penetration in the moderately cold regions. The asphalt is classified as a "standard grade" asphalt. The other, the "special grade" asphalt requires a higher penetration and a minimum PVN of –0.2. This grade is specified for cold regions, such as Interior Alaska and Greenland. It should be noted that both the standard and the special grades have the same stiffness at 60°C.

From his field studies, McLeod (1978) concluded that a well-designed and constructed dense-graded asphalt concrete mix will show low temperature transverse cracking when the asphalt mixture stiffness exceeds 6895 MPa (10^6 psi) for a 20,000-second loading time at some critical temperature in the area. This stiffness is reached due to a combi-

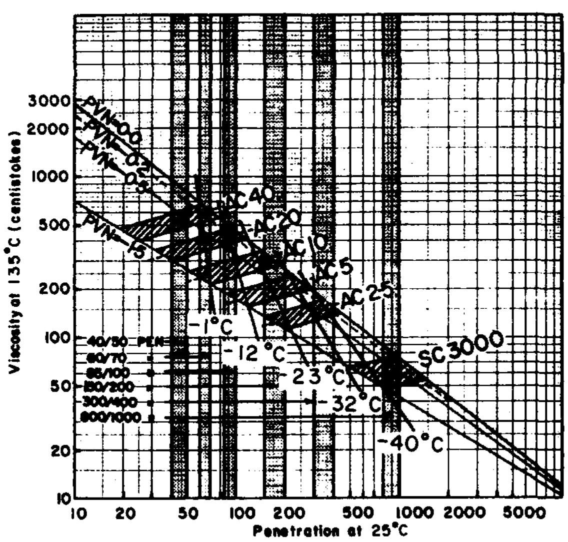

Figure 20. Guide for selection of asphalt for pavements in cold regions. To minimize low temperature contraction cracking during the pavement's service life, the grade of asphalt selected should lie on or to the right of the temperature-labeled diagonal line that indicates the minimum anticipated temperature at a pavement depth of 51 mm for surface courses during their service lives, or that indicates the minimum temperature expected within a base or binder course layer during its service life. Criteria for other design temperatures may be determined by interpolation.

nation of asphalt hardening and cold temperature. As expected, different asphalt grades will reach the minimum stiffness at different temperatures. The harder asphalts will reach this minimum stiffness at a higher temperature than their softer counterparts, leading to earlier low temperature cracking.

Figure 20 is used by the Corps of Engineers for selecting asphalt for minimizing low temperature cracking. This figure was originally developed by McLeod (1975). For minimizing low temperature cracking, the grade of asphalt selected should be to the right of the minimum anticipated temperature diagonal line. The −40° and −23°C temperature lines in the figure was based on field data obtained from Ste. Anne test road and the four Ontario test roads respectively. The minimum temperatures used in the figure are obtained approximately 51 mm below the pavement surface. Surface air temperatures can be used with this figure as the error in the stiffness calculated using the surface temperature is small (Carpenter and VanDam 1985).

At present, these specifications do not address the problem of rutting directly. It is assumed that if the stability and flow values from the Marshall mix design were adequate, rutting was not going to be a problem. However, as stated earlier, there is no correlation between Marshall stability and rutting. Brown (1987) suggested that the Marshall test be used only for determining the asphalt content.

PAVEMENT SURVEYS

Most of our knowledge on the use of soft asphalts come from Alaska and Canada. CRREL initiated two field surveys on the use and performance of soft asphalt concrete pavements in the United States. The first survey focused on the use of temperature susceptibility specifications by other Federal and State agencies. The second survey studied the low temperature cracking performance of soft asphalts and their performance with respect to thermal cracking and rutting within the states.

Airfield pavement survey

The first CRREL survey, conducted in 1984, was reported by Carpenter and VanDam (1985). They contacted other Corps of Engineers offices, State DOTs and State departments of aviation in areas

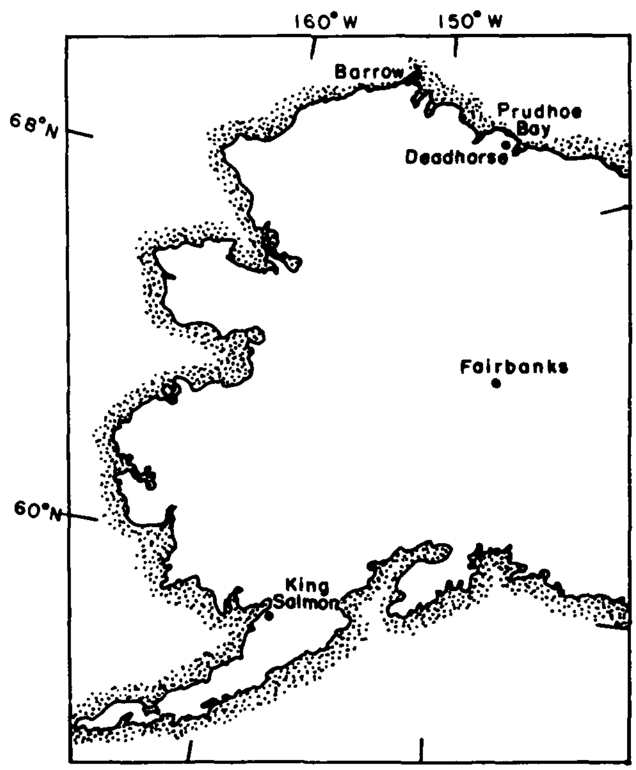

Figure 21. Location of airfield sites in Alaska.

where they thought specifications similar to those used by the Corps might be used. The states contacted were Alaska, Maine, Minnesota, North Dakota and Montana. They found several airfields in Alaska that met the Corps requirements for temperature susceptibility. The airfield sites examined were at King Salmon, Barrow, Deadhorse and Fort Wainwright (see Fig. 21) between 19 and 25 June 1984.

At King Salmon airfield (military and civilian), AC 5 asphalt with a PVN of –0.2 was used in a full depth replacement of part of the main runway, and on new construction on the ramps at the alert hanger. Construction took place between July and August 1981. After three years of service, the ramp and apron showed no signs of thermal cracking. Also there was no indication of rutting from channelized wheel loadings on the apron. However, the level of traffic on the apron was unknown. The report did not indicate whether thermal cracks were seen on the reconstructed runway. Pekar* (1987) indicated that there were no signs of thermal cracks on the runway as of July 1987.

At Barrow, the civilian airfield consisting of a runway and two taxiways was constructed between 1972 and 1974. A 5-mm-thick overlay was placed on the east taxiway and the runway in the summer of 1981 with AC 2.5 and 200/300 pen asphalt cement. The report does not say whether the asphalt cement met Corps specifications, and all indications are that it did not. Carpenter and VanDam (1985) observed that the taxiway and the runway looked no different from the apron that had not been overlaid. They reported large transverse cracks at an average spacing of 13.7 m. The cracks were considered to have reflected through the overlay. Fine cracks that ran transversely at an average spacing of 4.6 m and longitudinal cracks were also reported. Rutting was not reported although the report stated that the mixture consisted primarily of rounded aggregates and the contractor was having problems meeting the minimum 227-kg stability requirements.

The airfield at Deadhorse was constructed by an oil company in 1968. In 1978, the original gravel runway was widened and thickened with a 7.5-cm-thick asphalt concrete layer. The asphalt cement used was an AC 2.5 with a PVN of –0.6 and a penetration of 258. Carpenter and VanDam (1985) reported severe cracking on the runway and

* J. Pekar, Alaska District, COE, Personal comm., 1987.

Figure 22. Severe earth-related cracking in runway at Deadhorse (Carpenter and VanDam 1985).

taxiway. These cracks were considered to be due to the natural movement of ice wedge polygons under the pavement. Extremely low temperatures in the tundra caused cracks to form in and above the ice wedges. These cracks often extended upward through the pavements. This form of cracking is considered to be independent of the type of asphalt used and is shown in Figures 22 and 23. Carpenter and VanDam (1985) reported fine thermal cracks due to asphalt brittleness spaced between 1.2 m and 2.7 m on the runway.

The airfield at Fort Wainwright located near Fairbanks, Alaska, recently underwent major reconstruction. In 1983, part of the South Taxiway and Taxiway 3 was reconstructed with Special Grade (PVN = –0.2) AC 2.5. A plan of Fort Wainwright Airfield showing the recent repairs and the grade of asphalt used is shown in Figure 24. A year later, the reconstructed portion of the South Taxiway between Taxiway 3 and 4 showed fine low severity transverse cracking over the entire length (Carpenter and VanDam 1985). The crack spacing varied from 31 m to 62 m with an average

Figure 23. Severe earth-related cracking in granular fill at Deadhorse (Carpenter and VanDam 1985).

of 38 m. Taxiway 3 also showed fine low severity transverse cracking with spacing of approximately 31 m.

The author revisited Fort Wainwright in 1987 and made a visual survey of the runway and taxiway for low temperature cracks and ruts. Nearly all the cracks were sealed. On the South Taxiway, there were 20 transverse cracks over the entire length of 1128 m, averaging approximately 61 m apart. It was concluded that no new cracks were formed as the number of cracks in this area is similar to that reported by Carpenter and VanDam (1985). On Taxiway 3, 15 cracks were observed over a 640-m length and the crack spacing

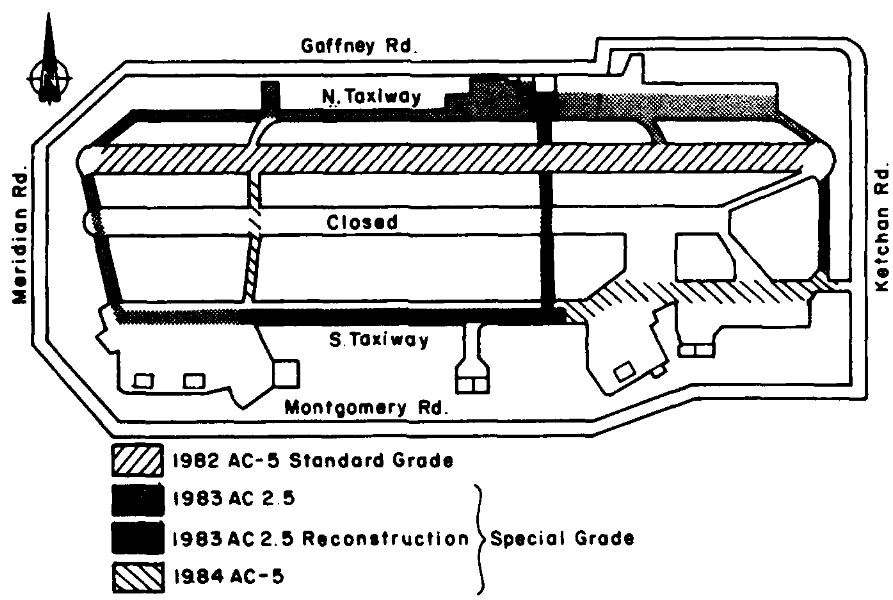

Figure 24. Location of COE-specified asphalt in Fort Wainwright Airfield complex.

Table 11. Calculated maximum and minimum temperature (°C) variation with depth (Carpenter and VanDam 1985).

	Depth (51 mm)	
	Maximum	*Minimum*
Barrow	19.2	-33.2
Fairbanks	40.5	-31.6
King Salmon	30.3	-17.3
Anchorage	34.3	-17.1
International Falls	45.5	-23.2
Great Falls	46.1	-15.2
Caribou	40.4	-18.5
Chicago	44.2	-11.6

was on the average about 43 m. This is again similar to that reported by Carpenter and VanDam (1985). Taxiway 4 was constructed with Arctic Grade AC 5 in 1984. After three years of service, there were four transverse cracks over the entire length of 482 m. This averages to approximately one crack every 122 m. Rutting was not seen at these sites.

In summary, the AC 2.5 and AC 5 asphalt used at King Salmon and Fort Wainwright performed well in controlling low temperature cracking. It is also evident from the survey that thermal cracking can also be caused by factors other than asphalt stiffness, as encountered in Deadhorse. Research in West Texas (Carpenter and Lytton 1972) indicated that thermal action occurs throughout the pavement structure. The results of this research showed that even under mild temperature fluc-

Table 12. Stiffness (MPa) for minimum temperatures for different asphalt cement mixtures, 51-mm depth (Carpenter and VanDam 1985).

Asphalt	*PVN*	*Pen*	*Barrow*	*Fairbanks*	*King Salmon*	*International Falls*	*Great Falls*	*Caribou*
Cold regions grades								
AC-2.5	–0.2	290	1724	1379	110	317	69	138
AC-5	–0.2	180	3654	2465	276	758	193	200
Conventional grades								
AC-10	0.0	200	2758	2206	214	572	145	262
AC-5	–0.5	200	3861	3034	255	689	165	310
AC-5	–1.0	200	5309	4137	290	896	193	359
AC-2.5	–1.5	110	6412	5861	4275	6826	579	965
AC-20	0.0	110	5929	5102	641	1655	434	758
AC-10	–0.5	110	7584	6481	758	2068	496	896
AC-10	–1.0	110	9653	8274	2551	2551	552	1034
AC-5	–1.5	110	13790	11721	1034	3447	676	1310
Alaskan asphalts								
AC-2.5-DH	–0.6	258	2827	2137	159	469	103	200
AC-2.5-W	+0.5	378	758	593	52	152	35	63
AC-5-W	–0.2	255	2137	1724	145	400	90	179

Table 13. Comparison of asphalt selection based on analytical method and Corps of Engineers specificiations.

Location	*Asphalt grade*	*Penetration*	*PVN*	*Mixture stiffness (MPa)*	*Asphalt grade*	*Penetration*	*PVN*	*Mixture stiffness (MPa)*
International Falls, Minnesota	AC–10	110	–1.0	2551 @ –23°C	AC–10	150	–0.5	517 @ –23°C
Great Falls, Montana	AC–5	110	–1.5	676 @ –15°C	AC–10	110	–0.5	496 @ –15°C
Caribou, Maine	AC–5	110	–1.5	1310 @ –19°C	AC–10	120	–0.5	689 @ –17°C

tuations, the level of thermal activity was sufficient to crack the base course and extend a crack upward through the asphalt concrete surface. It was also seen that reflection cracking cannot be controlled by the use of soft asphalts.

In their report, Carpenter and VanDam (1985) speculated on the use of PVN graded AC 2.5, AC 5 and AC 10 asphalt in the continental United States. They conducted an analytical comparison of asphalt concrete mixture stiffness at pavement depths 2.5 mm to 127 mm at International Falls (Minnesota), Great Falls (Montana) and Caribou (Maine). The pavement temperatures were calculated using the modified Barber's equation (Shahin and McCullough 1972). The minimum and maximum temperatures at 5 mm depth are presented in Table 11. The minimum temperatures were used to calculate the asphalt stiffness using a modified Van Der Poel's nomograph and then the mixture stiffness for different types of asphalt cements using the Heukelom and Klomp nomograph. The mixture stiffness calculated by Carpenter and VanDam was based on 3% air void and 14.5% VMA. The mixture stiffness at a depth of 51 mm is presented in Table 12. For selecting asphalt cements to minimize low temperature cracking, Carpenter and VanDam (1985) used a limiting criteria of 3448 MPa for 20,000 seconds of loading for low temperature cracking, instead of the criteria developed by McLeod (1976) of 6895 MPa for 20,000 seconds. They felt that using a lower stiffness value accounted for pavement aging. Based on this criteria, an AC 10 with a PVN of –1.0 would be adequate for controlling low temperature cracking in International Falls, Minnesota, and an AC 5 with a PVN of –1.5 would meet the requirements in Great Falls, Montana, and Caribou, Maine. Based on Figure 20 for a PVN of –0.5, the Corps of Engineers require an AC 10 asphalt for International Falls, Minnesota, an AC 10 for Great Falls, Montana and an AC 5 for Caribou, Maine (see Table 13). As indicated in Tables 12 and 13, by controlling the temperature susceptibility (PVN) of an asphalt, high viscosity asphalts can be used to minimize low temperature cracking.

Haas et al. (1987) presented results from a survey of 26 airports across Canada. He found that the two major problems affecting pavement deterioration were thermal cracking and stripping. With respect to thermal cracking, the following conclusions were presented. *The low temperature cracking seen at the Canadian Airports were significantly affected by the asphalt grade, the temperature susceptibility (PVN) of the asphalt, minimum temperature reached, coefficient of thermal contraction of the mix and thickness of the asphalt concrete layer.* Haas et al. (1987) concluded from their airport study that a potential saving of $10 million (1985 Canadian dollars) could be achieved if thermal cracking were reduced by 50% in the Canadian Air Transportation Administration (CATA) system.

Vinson et al. (1987) conducted a survey of over 200 general aviation airports in the United States. One of the purposes of the survey was to identify problems in airfield pavements that are distinctive in cold regions. The survey concluded that the dominant problem was cracking (low temperature and reflection cracking). The study further suggested that substantial benefit could be obtained by introducing additives to the asphalt or the asphalt concrete mixture for minimizing low temperature cracking. Further, the study stated that field techniques for controlling cracking were lacking.

Highway pavement survey

This field study looked at the type of asphalts, aggregates characteristics and mix design used by State DOTs in cold regions and whether low temperature cracking and rutting were a major problem in these states. This study was conducted with the assistance of State DOTs and the Alaska District, Corps of Engineers. Twenty-seven State DOTs were contacted. These states were Maine, New Hampshire, Vermont, Massachusetts, Connecticut, New York, Pennsylvania, Ohio, Michigan, Illinois, Indiana, Wisconsin, Minnesota, North Dakota, South Dakota, Nebraska, Iowa, Kansas, Colorado, Utah, Wyoming, Montana, Idaho, Nevada, Oregon, Washington and Alaska. Some site visits were made to evaluate roads constructed with soft asphalt binders.

It is the author's experience that the word "soft" used with asphalt is relative. In the Northeast, any asphalt whose absolute viscosity is less than 200 Pa•s (AC 20) at 60°C is considered a soft asphalt. In the state of Michigan, any asphalt whose penetration grade at 25°C is lower than 150/200 (equivalent to AC 5) is considered hard. In Alaska, any asphalt whose absolute viscosity at 60°C exceeds 25 Pa•s (AC 2.5) is considered hard. In this report, soft asphalts will be defined as any asphalt whose absolute viscosity is 100 Pa•s (AC 10) or less.

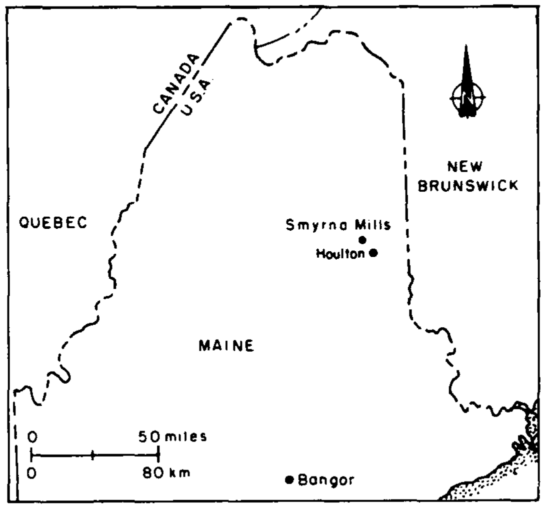

Figure 25. Location of test sections in Houlton, Maine.

Maine Department of Transportation

The Research Division of Maine DOT studied the influence of two grades of asphalt on low temperature cracking. The test site, built in 1977, was located between Smyrna and Houlton in the northbound lane. The location of this site is shown in Figure 25. The total length of the test site was 15.4 km. The pavement consisted of three layers with different binders used in the layers. The structure of the pavement is shown in Figure 26. The asphalt concrete grades were AC 5 and AC 10. The complete AC 5 test section was 4.5 km long and the complete AC 10 test section was 2.6 km. The penetration and viscosities of the asphalt used in the test section are presented in Table 14. The results presented here were reported in an interim report by Pilsbury and Jacobs (1985) and Rand (1987). The amount of thermal cracking developed between 1977 and 1985 is presented in Figure 27. As seen in the figure very little cracking was seen during the first three winters in both test sections. During 1980–81 winter, the AC 5 developed 49 m of cracks/km and the AC 10 section developed only 32 m of cracks/km. However, in the next four years, the amount of cracking in the AC 10 test

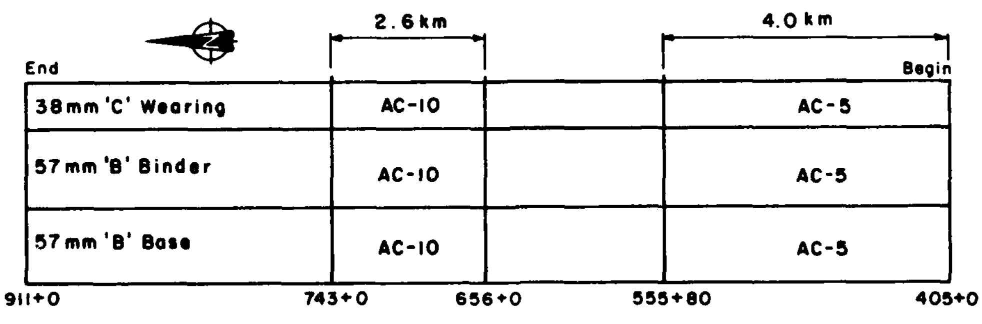

Figure 26. Pavement structure of test sections in Houlton, Maine (Pilsbury and Rand 1985).

Table 14. Viscosities, penetration and PVN values.

Asphalt	Visc @ 60°C Pa·s	Visc @ 135°C $m^2 \cdot s$	Pen @ 25°C dmm	PVN
AC 5	59	224×10^{-6}	167	−0.52
AC 10	92	289×10^{-6}	127	−0.29

section increased more rapidly than in the AC 5 test section. Also after 8 years of service, there was less cracking in the AC 5 test section than in the AC 10 test section. More cracks were observed in the passing than in the driving lane. One can only speculate as to why there was more cracks in the AC 5 test section early in the pavement life. One possible explanation may be due to the reduced temperature susceptibility of (PVN) the AC 10 (see Table 14). With time the AC 10 asphalt hardened at a faster rate than the AC 5 asphalt, thus producing more cracks in the AC 10 asphalt pavement. The minimum winter temperature in the Houlton area is around −20°C. The study concluded after 8 years that 1) there was 65% more thermal cracking in an AC 10 than in an AC 5 pavement and 2) thermal cracks appeared to be increasing at a higher rate in the AC 10 pavement.

During the same period, rut depth measurements were made. At the end of 8 years, rut depths of 6.8 mm were measured in the AC 5 asphalt and 7.4 mm in the AC 10 asphalt. Severe rutting was seen at intersections in Augusta, Maine, where the pavements are constructed with AC 20 asphalt. Maine DOT officials believe that the subgrade constructed 30 years ago was inadequate and suggest that most of the pavement rutting seen today in the city is due to rutting in the subgrade.

Another study conducted by Maine DOT (Foster 1987) analyzed 37 projects with 38-mm overlays in regard to grades of asphalt used and amount of daily 80-kN loads. They found that life expectancy of an AC 10 road was approximately 13 years and of an AC 20 road it was about 11 years. The results with respect to serviceability index are presented in Figure 28. The results suggest that using soft asphalts, in this case an AC 10 in overlays, may actually retard the rate of reflection cracking because AC 10 is more compliant than the AC 20.

Alaska DOT

Several studies were conducted by Alaska DOTPF on asphalt concrete properties and performance in Alaska (McHattie 1981, Henry 1981 and Osterkamp et al. 1986). Based on results from core samples (12 to 14 years) from 117 test sites throughout Alaska, McHattie (1981) concluded that the better performing pavements with respect to thermal cracking and rutting after 12 years had the properties outlined in Table 15. He also concluded that 80% of the originally specified penetration is lost within seven years and suggested that because of the nonlinearity in the aging process, performance assumption of pavements cannot be made from original asphalt properties. McHattie (1981) further suggested performance assumptions be based on age-hardened asphalts.

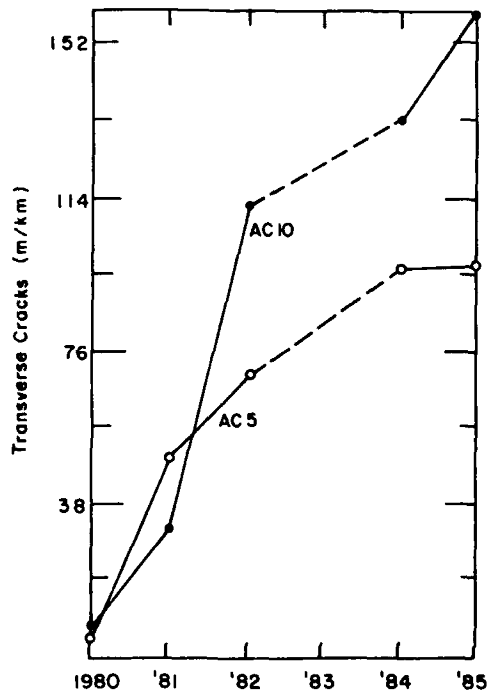

Figure 27. Annual amount of low temperature cracking in test sections in Houlton, Maine (Rand 1987).

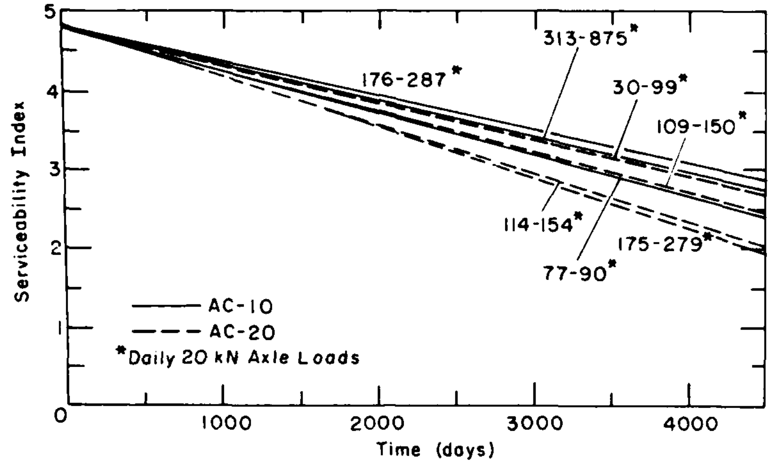

Figure 28. Influence of asphalt cement grade on pavement serviceability index (Foster 1987).

Another study conducted by Alaska DOTPF was to determine the causes of paving problems in Alaska in the early seventies. Some of the problems reported were laydown, flushing, cracking, surface abrasion and ravelling. Similar problems were reported by New York State about the same time. Henry (1981) reported on 31 Alaskan paving projects on sites selected statewide that represented poor, good and normal performance conditions with respect to thermal and fatigue cracking, raveling and stripping, rutting and excessive wear. These sites were further divided into those constructed during the time when penetration grading system was used and the rest during the AR or AC grading system was used.

Henry (1981) concluded that changing from the penetration grading system to the AR or AC grading system produced varying asphalt penetrations at 25°C. This change resulted in very differ-

Table 15. Best long-term asphalt concrete properties.

Minimum AC pavement thickness	51 mm
Minimum recovered penetration (77°F)	40 dmm
Minimum recovered penetration (39.2°F)	15 dmm
Maximum recovered absolute viscosity	800 Pa·s
Minimum bitumen content	5.5%
Maximum passing #10	39%
Maximum passing #40	19%
Maximum passing #200	7%

Table 16. Low-temperature cracking survey results.

State/agency	*Is low temp cracking considered to be a serious problem ?*	*Any Provisions in specs for low temp cracking ?*
COE	Yes	Yes
Alaska	Yes	Yes
Colorado	Yes	Yes
Connecticut	No	No
Idaho	Yes	Yes
Illinois	No	No
Indiana	No	No
Iowa	Yes	Yes
Kansas	Yes	Yes
Maine	Yes	Yes
Massachusetts	No	No
Michigan	Yes	Yes
Minnesota	No	Yes
Montana	Yes	Yes
Nebraska	Yes	Yes
Nevada	Yes	No
New Hampshire	Yes	Yes
New York	Yes	No
North Dakota	Yes	Yes
Ohio	Yes	No
Oregon	No	No
Pennsylvania	No	No
South Dakota	Yes	No
Utah	Yes	Yes
Vermont	Yes	Yes
Washington	Yes-East	Yes
Wisconsin	No	Yes
Wyoming	No	No

Table 17. Grade of asphalt cement specified.

State/agency	*Grade of AC specified*
COE	Based on procedure outlined in Fig. 20.
Alaska	AC 2.5 in Fairbanks. AC 5 and AC 10 in Anchorage and Juneau.
Colorado	AC 5 at elevations ≥ 2.4 km. AC 10 AC 20 in South and Southeastern Colorado.
Connecticut	AC 20
Idaho	AC 10
Illinois	AC 20
Indiana	AC 20
Iowa	AC 5 in county roads. AC 10 AC 20 in Interstates.
Kansas	AC 10 in the North. AC 20 in the South.
Maine	AC 10 north of Bangor. AC 20 elsewhere
Massachusetts	AC 20
Michigan	85/100 pen in the South. 120/150 and 200/300 pen in the North.
Minnesota	85/100 pen, 120/150 pen. 200/300 pen
Montana	85/100 pen
Nebraska	AC 10
Nevada	AC 20 or AR 8000
New Hampshire	AC 10
New York	AC 20
North Dakota	120/150 pen
Ohio	AC 20
Oregon	AC 20 or AR 4000
Pennsylvania	AC 20
South Dakota	60/70 pen
Utah	AC 10
Vermont	AC 10
Washington	AR 4000W
Wisconsin	85/100 pen in the South. 120/150 pen in the North.
Wyoming	AC 20

Table 18. Maximum penetration values specified.

State/agency	*AC grade*	*Max pen*
COE	See ETL 1110-3-369	
Colorado	AC 20	100
Indiana	AC 20	110 (50 MIN)
New York	AC 20	100
Vermont	AC 10	140
Pennsylvania	AC 20	120

ent handling and compaction properties of different asphalt mixes during the mid-seventies and was probably the main cause of the problems. He further concluded that consistency in the penetration values in the AC grading system has since been reestablished.

SURVEY RESULTS

The results of the survey presented here generally is applicable to interstates and high-volume traffic roads. Information on other type of roads is presented in this report when available. The results of the survey are presented in the following tables.

Low temperature cracking

As shown in Table 16, *18 out of the 27 states surveyed considered low temperature cracking to be a serious problem.* Some states such as New York consider it to be serious in the Adirondack region only and Washington considers it to be a problem in the Eastern part of the state. Other states such as Minnesota and Wisconsin do have low temperature cracking but consider it not to be as serious a problem. Colorado considers low temperature cracking to be a problem but regards rutting and raveling to be more serious. Utah found that changing their source of asphalt minimized low temperature cracking that was prevalent in the late sixties/early seventies period.

Sixteen states have some form of specification for minimizing low temperature cracking (see Table 16). Alaska, Kansas, Maine, Michigan, Wisconsin and, to a certain extent, Colorado use softer grades of asphalt in the colder regions of the state. Colorado also specifies asphalt based on elevation above sea level. The remaining nine states use AC 10 or 85/100 pen or 120/150 pen asphalt. The grades of asphalt commonly specified by the states for their interstate highways are presented in Table 17. Michigan also uses 200/250 pen asphalt on low volume roads. Oregon uses all three grading systems (AC, Pen and AR) because it gets its asphalt from several out of state sources. In 1989, Oregon plans to use the viscosity grading system in its specifications. The AR 4000W grade specified by Washington is similar to AASHTO mix described in Table 3, with the exception that the minimum residue penetration at 25°C is 40 and not 25 as specified by AASHTO. The other difference is the ductility test at 7.2°C required by Washington and not required by AASHTO. The minimum ductility

at 7.2°C is 10 cm. Minnesota now uses 85/100 pen asphalt on interstates, 120/150 pen asphalt in the north and secondary roads, and 200/300 pen asphalt on low volume roads. It will stop using 200/300 pen asphalt in 1989.

All the states with the exception of Pennsylvania and Oregon do not require the asphalts to meet any temperature susceptibility requirements. Pennsylvania uses the PVN concept for indexing temperature susceptibility. Their specifications use a sliding scale on the minimum kinematic viscosity requirement, based on the penetration at 25°C. For example, if the penetration of the AC 20 asphalt is 60 pen, then the minimum specified kinematic viscosity is 390 centistokes (cst). If the penetration is 70+, then the specified kinematic viscosity is 300+ cst. The temperature susceptibility of the asphalt generally used ranges between a PVN of –0.7 to –0.9. Oregon specifies that the minimum penetration at 4°C cannot be less than 25% of the penetration value at 25°C. Indirectly, temperature susceptibility specifications have been introduced by Vermont, New York, Colorado and Illinois by placing a cap on the penetration values (see Table 18).

Vermont also places a cap of 290 cst on the kinematic viscosity. With respect to PVN, the AC 10 asphalt supplied to Vermont ranges from –0.5 to –1.1. Instead of PVN or PI, Washington uses a low temperature ductility test (introduced in 1974) at 7.2°C for specifying asphalts. The minimum ductility at this temperature is 10 cm; 7.2°C was chosen instead of the common 4°C because of difficulty at running this test at the lower temperature. Nevada, however, requires a ductility test at 4°C performed on rubberized asphalt. Also, Utah requires a similar low temperature test when polymer-modified asphalts are used.

Rutting

With respect to rutting, it is known that factors such as gradation, fines content, asphalt content, amount of natural sand, amount of crushed aggregates and mix properties play a major role. Instead of just looking at the type of asphalt used, the characteristics of the aggregates used as well as the mix properties of the states were included in the survey. As shown in Table 19, *17 states considered rutting to be a serious problem*. Several states have introduced rut resistant mix design (see Table 20). From Table 17, *9 out of 17 states that reported rutting to be a serious problem use AC 20 asphalt cement*. Colorado and Oregon now require a Hveem stability of 37 for their mixes. To get a Hveem stability of 37, the aggregate used in the mix is almost 100% crushed and considered to be rut resistant. Washington also plans to increase the Hveem stability to 37 in 1989. Minnesota now uses 80–100% crushed aggregates for their wearing course and, depending on the traffic load, 50 to 70% crushed aggregates in the no. 4 sieve in the binder course. There is no limit on the natural sand in the binder course. Utah uses PCC pavements in high density traffic areas and is currently experimenting with an AC 20 modified with latex asphalt in the mix for minimizing rutting. Indiana is experimenting with large stone open-graded (63.5 mm top size) mixes for minimizing rutting.

The results of the survey on the aggregate characteristics and mix properties are tabulated in Tables 21 to 31. With respect to aggregates, the standard L.A. Abrasion test is used by nearly all the states with maximum allowable loss ranging from 30% to 50% for the coarse aggregates. Michigan, Minnesota and Vermont require the same test on the fine fraction (Table 21). About half the states

Table 19. Severity of rutting.

State/agency	*Is rutting considered to be a serious problem ?*
COE	N/A
Alaska	Yes, in Anchorage and Juneau.
Colorado	Yes
Connecticut	No, but localized rutting seen.
Idaho	Yes, but localized.
Illinois	No
Indiana	Yes
Iowa	No
Kansas	Yes
Maine	No, consider ruts >13 mm as serious.
Massachusetts	No
Michigan	No
Minnesota	Yes, consider ruts >13 mm as serious.
Montana	Yes
Nebraska	Yes
Nevada	Yes
New Hampshire	No
New York	Yes
North Dakota	No
Ohio	Yes
Oregon	No
Pennsylvania	Yes
South Dakota	Yes
Utah	Yes
Vermont	Yes, mainly at intersections.
Washington	No
Wisconsin	Yes
Wyoming	Yes

Table 20. Provisions for rutting.

State/agency	*Provisions for rutting ?*
COE	Yes
Alaska	No
Colorado	Yes, mix is considered to be rut resistant if it has a Hveem stability of 37.
Connecticut	No
Idaho	No
Illinois	Yes, has rut resistant mix design for PCC overlays.
Indiana	No
Iowa	Yes
Kansas	Yes
Maine	No
Massachusetts	No
Michigan	No
Minnesota	Yes
Montana	No
Nebraska	Yes, specified as Type A Special.
Nevada	Yes
New Hampshire	No
New York	No
North Dakota	No
Ohio	No
Oregon	Yes, WASHTO anti-rut mix, Hveem stability of 37 and lime treated base.
Pennsylvania	Yes
South Dakota	No
Utah	Yes, AC20R.
Vermont	No
Washington	No
Wisconsin	Yes
Wyoming	Yes, WASHTO guidelines.

surveyed require soundness tests on the coarse aggregates and only about a third require the test on the fine aggregates (Table 22). Some states such as Colorado, Iowa and Kansas require freeze-thaw testing on the fine portion. All the states with the exception of Colorado (Table 23) require crushed coarse aggregates. Many of the states that reported rutting to be a serious problem have implemented crushed aggregate requirements. Iowa, Minnesota, Nevada, Oregon, Pennsylvania, Wisconsin and Wyoming limit the amount of natural sand in the mix to 25% (Table 24). Massachusetts and Utah use 100% crushed aggregates in their mix. Other states such as New Hampshire, North Dakota, Ohio and Vermont require a minimum of 50% natural sand in the mix for workability. This I believe will be the cause of more rutting in the future in these states.

With respect to mix design, nearly all the states required higher stability requirements than the COE (Table 25). The flow, design air voids and VMA (Tables 26, 27, 28) specified by the states follow the Asphalt Institute guidelines. Some states are increasing their design air voids (4–6%) for anti-rut mixes. Besides the COE only Alaska uses the Voids Filled (VFA) criteria. I believe that the VFA is a better indicator for the rutting potential of a given mix as it reflects the volume of asphalt in the voids. The asphalt content used by the states are determined from the Marshall or Hveem procedure. Typical ranges are presented in Table 29. Most of the states that use the Marshall mix design method are compacting their specimens with 75 blows (Table 30). I believe that the COE should also consider changing from 50 to 75 blows. As shown in Table 31 many states also specify mat density as a percentage of the density obtained from the theoretical maximum specific gravity (ASTM 2041-78, also known as the Rice Method).

Table 21. Coarse and fine aggregates—L.A. abrasion maximum allowable % loss of original weight.

	Maximum allowable % loss after 500 rev.	
State/agency	*Coarse*	*Fine*
COE	40	40
Alaska	45	No
Colorado	45	No
Connecticut	40	No
Idaho	35	No
Illinois	40	No
Indiana	40	No
Iowa	45	No
Kansas	40	No
Maine	No	No
Massachusetts	30	30
Michigan	40	40
Minnesota	40	No
Montana	No	No
Nebraska	40	No
Nevada	45	Dense-graded mix + #12
	37	Wearing course mix
New Hampshire	No	No
New York	35	No
North Dakota	40	No
Ohio	40	No
Oregon	30	No
Pennsylvania	40	No
South Dakota	40	No
Utah	No	40
Vermont	34	34
Washington	30	No
Wisconsin	50	No
Wyoming	40	No

Table 22. Coarse and fine aggregates soundness requirement.

	Maximum allowable % loss after 5 cycles			
	Coarse		*Fine*	
State/agency	*$MgSO_4$*	*$NaSO_4$*	*$MgSO_4$*	*$NaSO_4$*
COE	17–10 cycles		17–10 cycles	
Alaska	9		No, freeze-thaw test	
Colorado	No		No	
Connecticut	10		No	
Idaho	No		No	
Illinois	15		15 for surface course 20 for binder course	
Indiana	12		No	
Iowa	No		No, freeze-thaw test.	
Kansas	No		No, freeze-thaw test	
Maine	No		No	
Massachusetts	No		No	
Michigan	5		No	
Minnesota	No		No	
Montana	No		No	
Nebraska	12		No	
Nevada		12		10
New Hampshire	No		No	
New York	18–10 cycles		45	
North Dakota	No		No	
Ohio		15		No
Oregon		12		12
Pennsylvania		10		10
South Dakota	12		12	
Utah	No		No	
Vermont		7		7
Washington	No		No	
Wisconsin		Yes		No
Wyoming	No		No	

Iowa uses a percentage of the Quality Index, which is patterned after method R9-86 of AASHTO. Basically, seven cores are taken and the densities calculated. The mean of the seven densities is then compared with laboratory Marshall (75-blow) specimen density. The Quality Index is the difference between the mean and the Marshall lab density divided by the standard deviation of the field densities. Indiana specifies its minimum acceptable density as a percentage of densities obtained from a controlled test strip. New York has no minimum density requirements. It uses the number of passes method.

It can be seen from this survey that in the northern states, where rutting was a problem, most states did not change the grade of asphalt as a rut resistant measure. Many of the states have implemented more strict crushed aggregate requirements, minimizing the amount of natural sand in the mix and changes in mix design parameters. Some of these changes may create excessive low temperature cracking. For example, Wisconsin is now experimenting with high inplace air voids (7%). Unless a minimum asphalt film thickness is implemented such as Iowa has done, the mix may be very lean and brittle. Many of the states surveyed are implementing the new FHWA Technical Advisory (FHWA T5040.27) on asphalt concrete mix design and field control in their specifications.

Table 23. Coarse and fine aggregates—fractured face requirements.

State/agency	Minimum 1 fractured face requirements (% by weight) Coarse	Fine
COE	75	90–2 face on #30 sieve
Alaska	70	No
Colorado	No	No
Connecticut	50	No
Idaho	60–2 face	No
Illinois	100	50
Indiana	95–100	No
Iowa	100	
Kansas	50% of total mix 80% of total mix when used over PCC	
Maine	60–2 face	No
Massachusetts	100% crushed aggregates	
Michigan	60	No
Minnesota	80–100% crushed aggregates for wearing course	
Montana	70	No
Nebraska	100	65
Nevada	50	90% manufactured sand
New Hampshire	50	No
New York	75, +38 mm 85–2 face, –38 mm	No
North Dakota	95 on +16 mm	No
Ohio	40	No
Oregon	75–2 face	85% manufactured sand
Pennsylvania	85–2 face	75% manufactured sand
South Dakota	50	No
Utah	100% crushed aggregates	
Vermont	50	No
Washington	75 + #10	No
Wisconsin	90; 60–2 face	90% manufactured sand
Wyoming	100	80% manufactured sand

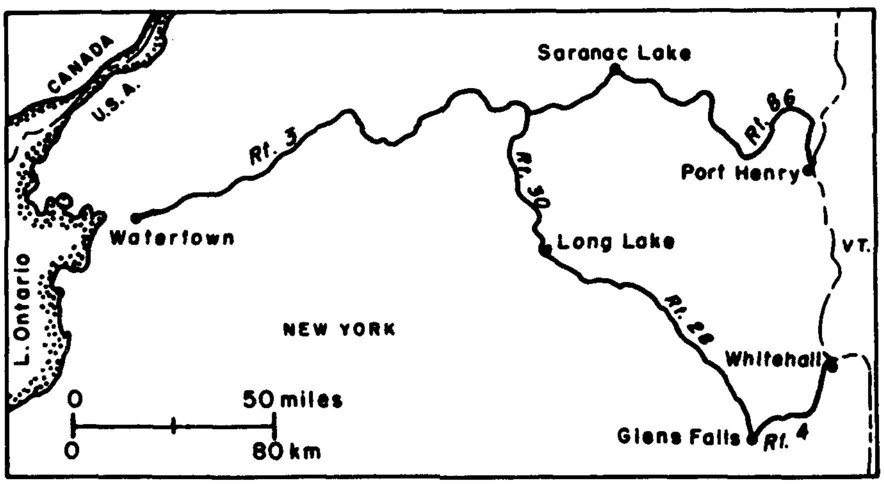

Figure 29. Survey route in upstate New York.

Table 24. Limit of natural sand in mix.

State/agency	*Natural sand limit (%)*
COE	25 75% manufactured sand
Alaska	No
Colorado	No
Connecticut	No
Idaho	No
Illinois	50
Indiana	No
Iowa	15
Kansas	min. 10–15% for workability typically 20–25% max. 20% in overlays on PCC
Maine	No
Massachusetts	0
Michigan	No
Minnesota	20
Montana	No
Nebraska	35
Nevada	10
New Hampshire	50 min. for workability
New York	No
North Dakota	50 min. for workability
Ohio	50 min. for workability
Oregon	15
Pennsylvania	25
South Dakota	No
Utah	0
Vermont	50 min. for workability
Washington	No
Wisconsin	10
Wyoming	20

Site visits

New York

The State of New York, with regards to the wearing surface, currently specifies AC 20 or 85/100 pen asphalt for the whole state. In western New York State, both AC 20 and AC 15 are used (supply of AC 15 is abundant in this region). In northern New York, 85/100 is also used as the source of asphalt in this area is usually Canada. The state basically uses a slightly modified version of AASHTO Table 2 for specifying asphalt. New York's modification of AASHTO Table 2 is to place a limit on the penetration value. Based on the penetrations and kinematic viscosity requirements, the PVN ranges from −0.7 to −1.1. At one time, the AC 10 was specified for upstate New York, but it was left to the Districts' discretion for enforcing the specifications. At present, the specifications have been changed from an AC 10 to an AC 20 for the top course, with the exception for resurfacing of rural, suburban and urban arterial roads in upstate New York.

A field survey of some 483 km of pavements in upstate New York showed extensive thermal cracking. The route sampled is shown in Figure 29. Some typical cracked pavement sections are shown in Figures 30 and 31. The New York DOT realizes that thermal cracking occurs in upstate New York, especially in the Adirondack Mountain regions, but believes that the pavement performance has not been affected by the presence of these cracks.

Figure 30. Typical low temperature cracking on Route 9N, New York.

Table 25. Minimum stability requirements and typical pavement structure.

State/agency	Minimum stability and typical pavement structure (N)			
	Base	*Binder*	*Wearing*	*Full depth*
COE	405	112	112	
Alaska	No	No	337	
Colorado (Hveem)				37
Connecticut	No	337	337	
Idaho (Hveem)				37
Illinois	No	450	450	
Indiana	270	270	270	
Iowa	No	393	393	
Kansas				450
Maine (Hveem)	25	25	25	
Massachusetts	Yes	Yes	Yes	—No minimum stability requirements
Michigan	Yes	Yes	337	
Minnesota	112	112/169	225	
Montana		—No minimum stability requirements	Yes	
Nebraska	Yes	No	Yes	—No minimum stability requirements
Nevada (Hveem)	37	37	37	
New Hampshire	Yes	Yes	225	
New York	Yes	Yes	337	
North Dakota	112	No	112	
Ohio	Yes	247	247	
Oregon (Hveem)	37	37	37	
Pennsylvania	483	483	483	
South Dakota	337	No	337	
Utah	270			
Vermont	225	225	225	
Washington (Hveem)	35	37		
Wisconsin	337	337	337	
Wyoming			405/35	

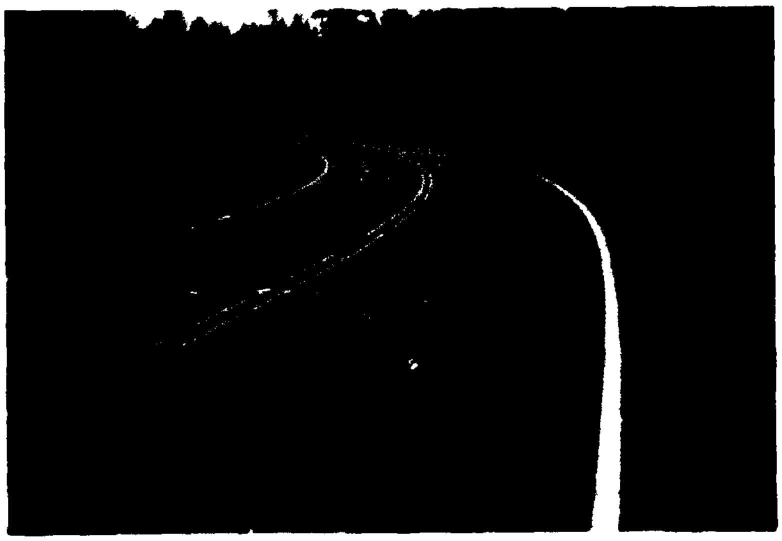

Figure 31. Typical low temperature cracking on Route 3, New York.

Table 26. Minimum flow requirements.

State/agency	Minimum flow requirements (%) Base	Binder	Wearing	Full depth
COE	16	20	20	
Alaska			8–16	
Colorado (Hveem)				N/A
Connecticut	No	8–15	8–15	
Idaho (Hveem)				N/A
Illinois	No	8–16	8–16	
Indiana	6–16	6–16	6–16	
Iowa	No	8–16	8–16	
Kansas				No
Maine (Hveem)	No	No	No	
Massachusetts	No	No	No	
Michigan	No	No	11–13	
Minnesota	No	No	No	
Montana				No
Nebraska	No	No		
Nevada (Hveem)	N/A	N/A	N/A	
New Hampshire	No	No	>11	
New York	No	No	8–18	
North Dakota	8–18	8–18		
Ohio	No	8–16	8–16	
Oregon (Hveem)	N/A	N/A	N/A	
Pennsylvania	6–16	6–16	6–16	
South Dakota	No	No	No	
Utah	10–18			
Vermont	8–20	8–20	8–20	
Washington (Hveem)	N/A	N/A		
Wisconsin	8–16	8–16	8–16	
Wyoming	8–16			

When New York State switched from penetration-graded to viscosity-graded asphalts in 1974, AC 10 asphalt was specified for the whole state. The pavements were found to be flushing and rutting, and the asphalt mixture was unstable during compaction. Flushing and unstable mix suggests that the mix may have contained too much asphalt. The asphalt properties may have been changed when the asphalt grading system was changed from penetration to viscosity grading, resulting in incorrect blending of asphalt cements at the refineries to meet the new specifications. The state had to lower its compaction temperature when it used the new AC 10 material. Another major concern of the state DOT was that the AC 10 paved lane could not be opened to traffic as quickly as an AC 20 lane. The lowering of the compaction temperature and slow setting nature of the asphalt suggests a high penetration low viscosity asphalt. A similar problem was seen by the paving contractor at Fort Drum in 1987. Initially, the contractor had problems with laying down the AC 10 asphalt mix (viscosity of a 1000 poise at 60°C and a penetration of 150) but with time was able to rectify these problems by reducing the breakdown and compaction temperature and changing the rolling pattern. To assess the benefits of soft asphalts, New York State conducted an informal study on the performance of AC 10 and AC 20 asphalt pavements and it was concluded that there was no significant difference in performance with respect to thermal cracking.

The New York DOT does not believe that rutting is a serious problem in the state. However, severe rutting at intersections was seen at Watertown, New York (where heavy truck traffic has increased due to major construction at Fort Drum). These pavements were constructed with AC 20. The District Office at Watertown is planning to experiment with a coarser gradation in their surface mix to try to reduce rutting. As for the aggregate, the coarse aggregates are crushed and must

have a minimum of 85% two face crushed aggregate. There is no crush requirement on the fine portion and either natural or manufactured sand is allowed in their mix.

Michigan

Michigan DOT specifies asphalt cement using either the viscosity or penetration grading system. The penetration grade is more commonly used. The pavement system in Michigan (1986) is approximately one-third rigid, one-third flexible and one-third flexible asphalt overlays over rigid pavements. The asphalt selection is based on Marshall stability and ADT as shown in Table 32. As seen in Table 32, the DOT specifies using 85/100 pen grade asphalt where the ADT in the right lane exceeds 400. In other areas 120/150 pen asphalt is recommended. However, the present trend north of Lansing and in the upper peninsula is to use 200/250 pen asphalt on primary and secondary roads*.

Michigan does not have a temperature suscep-

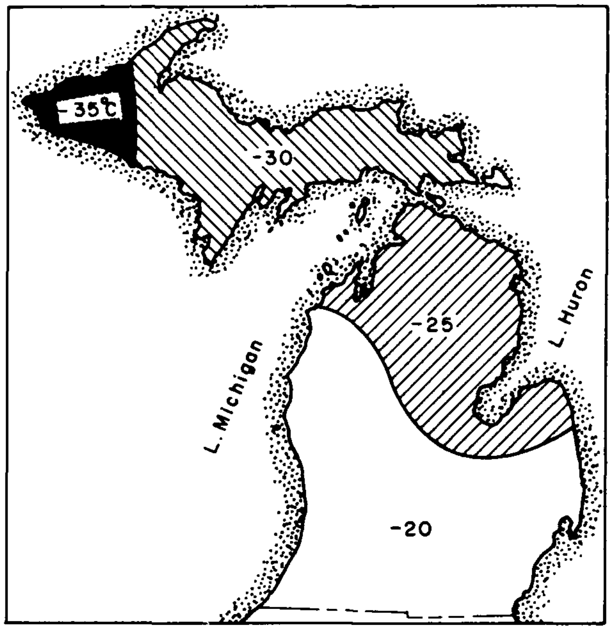

Figure 32. Winter design temperatures in Michigan (Novak 1976).

Table 27. Minimum design air void requirements.

	Minimum air void requirements (%)			
State/agency	*Base*	*Binder*	*Wearing*	*Full depth*
COE	4–6	4–6	4–6	
Alaska			1–5	
Colorado (Hveem)				4–6
Connecticut		3–6	3–6	
Idaho (Hveem)				3–5
Illinois	3–5	3–5		
Indiana	4–6	4–6	4–6	
Iowa	3.5–6	3.5–6	3.5–6	
Kansas (Hveem)				3–5
Massachusetts	No	No	No	
Michigan	No	2.5–3.5	2.5–3.5	
Minnesota	3–7	3–7	3–5	
Montana				3–5
Nebraska	No	No	No	
Nevada (Hveem)	3–6	3–6	3–6	
New Hampshire	No	No	3–5	
New York	No	No	2–4	
North Dakota	3–5	No	3–5	
Ohio	5.5	3–5	3–5	
Oregon (Hveem)	4–5	4–5	4–5	
Pennsylvania	4–6	4–6	3–5	
South Dakota	No	No	No	
Utah			2–4	
Vermont	3–5	3–5	3–5	
Washington (Hveem)	2–4.5		2–4.5 West 4–4.5 East	
Wisconsin	3–5	3–5	3–5	
Wyoming			2.5–3.5	

* Novak, E.C., Jr., Michigan Dept. of Transportation, personal communication, 1987.

Table 28. Minimum VMA requirements.

State/agency	Minimum VMA requirements (%) Base	Binder	Wearing	Full depth
COE	No	No	No	
Alaska	No			
Colorado (Hveem)				No
Connecticut	No	16	16	
Idaho (Hveem)				13
Illinois	No	13	14	
Indiana	13	15	16	
Iowa	No	13.5	13.5	
Kansas				No
Maine (Hveem)	No	No	No	
Massachusetts	No	No	No	
Michigan	No	No	15.5	
Minnesota	No	No	No	
Montana				13.5–15.5
Nebraska	No	No		
Nevada (Hveem)	14–22	14–22	14–22	
New Hampshire	No	No	No	
New York	No	No	15.5	
North Dakota	12–15		12–15	
Ohio	No	13	16	
Oregon (Hveem)	No	No	No	
Pennsylvania	10	11	12	
South Dakota	No	No	No	
Utah	No			
Vermont	13	14	15	
Washington (Hveem)	No	No		
Wisconsin	No	14	15	
Wyoming	15			

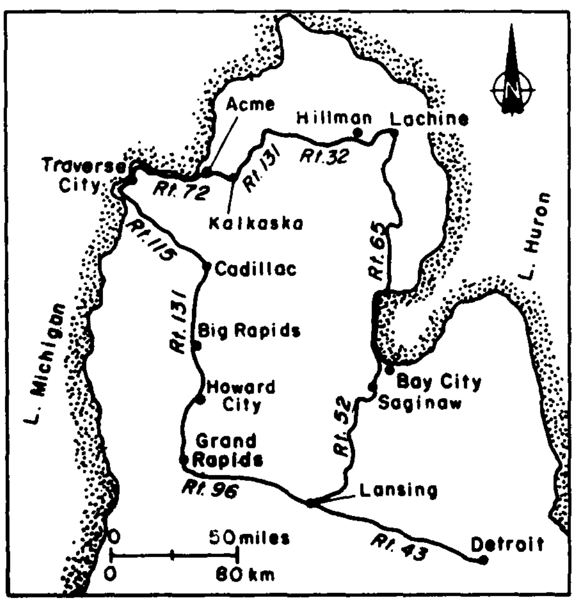

Figure 33. Survey route in Michigan.

tibility requirement on the asphalt. From available data (1985–1986), the PVN of the asphalts used in the state was found to range from −0.5 to −1.1. The Michigan DOT considers thermal cracking as a major problem and does not recommend using any asphalt harder than an AC 10. The winter design temperature for Michigan is shown in Figure 32. Michigan used 30/40 pen and 60/70 pen asphalt prior to the seventies. Because of low temperature cracking the grade was changed to 85/100 and softer asphalts. In areas where steep grades and/or heavy truck traffic exists, 85/100 is always used because of the increased shear force on the pavement.

The sand in the asphalt concrete mix is predominantly natural. At present there are no fracture face or crushed aggregate requirements on the fine portion of the mix. The pavements are beginning to show more rutting (Novak 1987). Michigan DOT does not believe that the rutting is

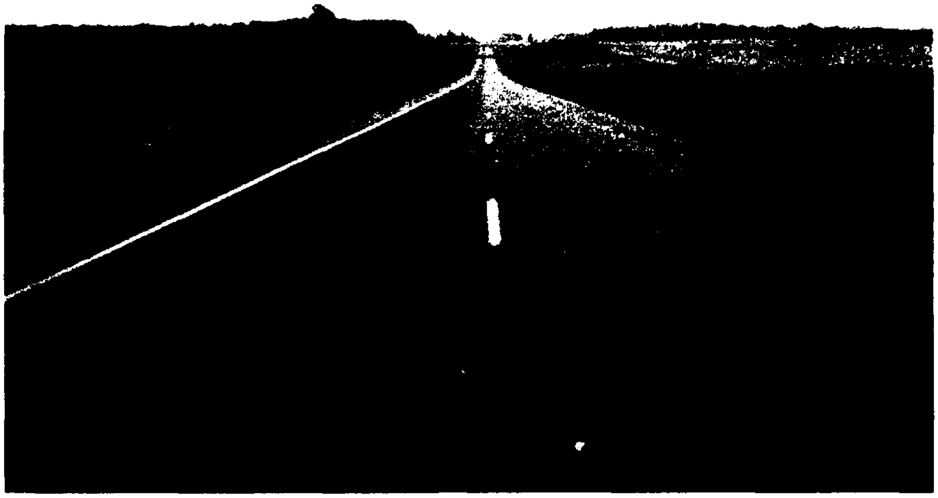

Figure 34. Typical flexible pavement section on Route 131, Michigan.

Table 29. Typical asphalt content.

	Typical asphalt content (%)			
State/agency	*Base*	*Binder*	*Wearing*	*Full depth*
COE	5.0–5.5	5.0–5.5	6.2–6.8	
Alaska			6.0–6.5	
Colorado (Hveem)				5.5–6.0
Connecticut	4.0 typ.	5.0–5.2	5.0–5.2	
Idaho (Hveem)				5.3–5.5
Illinois		4.5–5.0	5.0–6.0	
Indiana	4.3 typ.	4.5 typ.	6.0 typ.	
Iowa	4.5–5.0			
Kansas				5.0–6.0
Maine (Hveem)	5.0–5.5	5.0–5.5	6.2–6.6	
Massachusetts	4.0–5.0	4.5–5.0	5.5–6.5	
Michigan			4.0 typ.	
Minnesota	4.5–5.5	4.5–5.5	5.5–6.8	
Montana				5.8 typ.
Nebraska	3.9–5.5		4.4–5.5	
Nevada (Hveem)	4.0–8.5		4.0–8.5	
New Hampshire	3.8–4.8	4.8–6.0	6.0–7.0	
New York	4.0–6.0	4.5–6.5	5.8–7.0	
North Dakota				
Ohio	4.0–8.0	4.4–9.0	5.4 typ.	
Oregon (Hveem)			5.5–6.0	
Pennsylvania	3.5–6.0	3.5–6.0	3.0–6.0	
South Dakota			6.0 typ.	
Utah	5.5 typ.			
Vermont	4.0–6.0	5.0–7.0	5.5–8.0	
Washington (Hveem)	4.3–5.2		4.8–5.7	
	5.0–7.0	5.5–7.5 –basalt, ledge		
Wisconsin				
Wyoming			5.0–6.0	

Table 30. Number of Marshall compaction blows.

State/agency	Base	Binder	Wearing	Full depth
	No. of blows			
COE	75	50	50	
Alaska			75 urban	
			50 rural	
Colorado				
(Hveem)			N/A	
Connecticut		75	75	
Idaho				
(Hveem)				N/A
Illinois	50	75	75	
Indiana	75	75	75	
Iowa		75	75	
Kansas				50
Maine				
(Hveem)	N/A	N/A	N/A	
Massachusetts				
Michigan	50	50	50	
Minnesota	50	75	75	
Montana				50
Nebraska	50	50	50	
Nevada				
(Hveem)	N/A	N/A	N/A	
New Hampshire			50	
New York			50	
North Dakota	50	50	50	
Ohio	75	75	75	
Oregon				
(Hveem)	N/A	N/A	N/A	
Pennsylvania	75	75	75	
South Dakota	50	50	50	
Utah			75	
Vermont	50	50	50	
Washington				
(Hveem)	N/A	N/A	N/A	
Wisconsin	75	75	75	
Wyoming			75	

Table 31. Minimum mat density requirement.

State/agency	Minimum mat density (%)
COE	97% of Lab Marshall
Alaska	95% of Lab Marshall
Colorado	95% of Lab Hveem
Connecticut	92% of Theoretical Maximum Density
Idaho	92% of Theoretical Maximum Density
Illinois	93% of Theoretical Maximum Density
Indiana	98% of Controlled test strip density
Iowa	96% of Quality Index
Kansas	96% of Lab Marshall
Maine	93% of Theoretical Maximum Density
Massachusetts	96% of Theoretical Maximum Density
Michigan	97% of Field Marshall
Minnesota	91% of Theoretical Maximum Density
Montana	95% of 4 field Marshall Densities
Nebraska	92.5% of Theoretical Maximum Density
Nevada	96% of Lab Marshall
New Hampshire	95% of Lab Marshall
New York	No
North Dakota	97% of Lab Marshall
Ohio	92% of Theoretical Maximum Density
Oregon	91% of Theoretical Maximum Density
Pennsylvania	92% of Theoretical Maximum Density
South Dakota	95% of Lab Marshall
Utah	95% of Lab Marshall
Vermont	97% of Theoretical Maximum Density
Washington	92% of Theoretical Maximum Density
Wisconsin	92% of Theoretical Maximum Density
Wyoming	92% of Theoretical Maximum Density

Table 32. Selection of asphalt grade based on ADT and stability by Michigan DOT.

	Asphalt penetration (viscosity)	
Bituminous mix	Current commercial ADT in right lane	
	0–00	400 over
112–157 N stability Bituminous base	120–150 (AC 5)	85–100 (AC 10)
202–406 N stability Bituminous surface	120–150 (AC 5)	85–100 (AC 10)

due to the soft asphalt used but rather to the natural sand in the fine aggregate portion and high asphalt content. Typically, the asphalt content for the surface course runs between 5.5% and 6.1% with an average of 5.7%.

A field survey was conducted and the route surveyed is shown in Figure 33. Rutting 13 mm was seen in the city of Lansing, mainly at intersections. Rut depths at other locations along the route varied from a 0.64 to 1.27 mm. We were informed that Michigan DOT plans to overlay any pavement when the rut depth exceeds 9.5 mm. Truck traffic was light on our survey route. Several examples of roads constructed with soft asphalt were found along this route. A brief description of the condition of the roads is presented below. The survey was conducted in a slow moving vehicle.

The primary road surveyed, part of Route 131, outside of Howard City and Big Rapids, is approximately 5 to 7 years old and is approximately 32 km long. The pavement was constructed with 120/150 penetration grade asphalt. Very few thermal cracks were seen along this section. Rutting

Figure 35. Typical flexible pavement section east of Acme on Route 72, Michigan.

Figure 36. Typical flexible pavement outside Hillman on Route 32, Michigan.

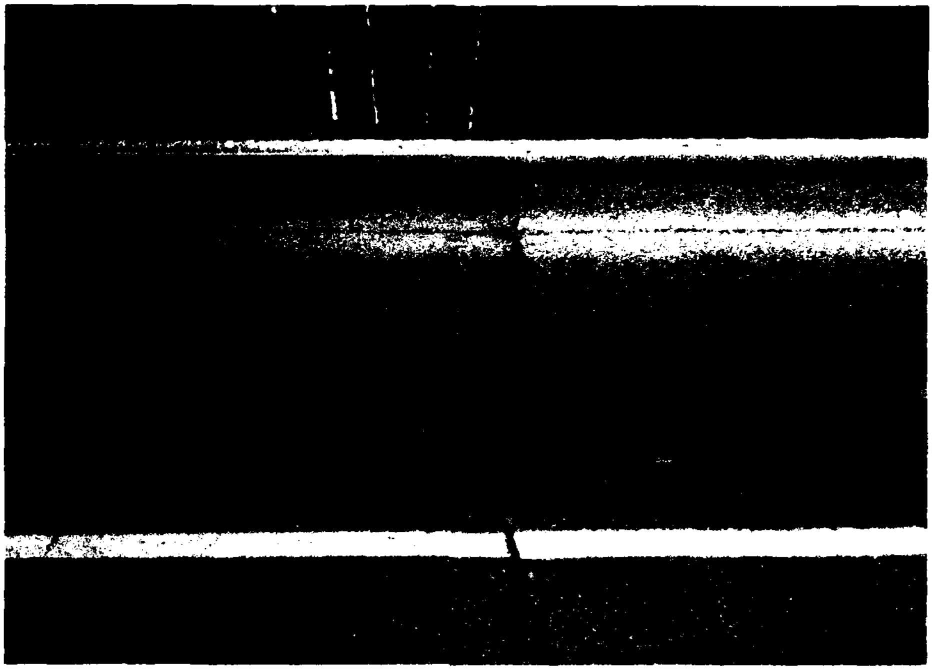

Figure 37. Typical low temperature cracking on Route 115, outside Cadillac, Michigan.

between 0.64 and 1.27 in. under the wheel path was observed. The road was considered to be in good condition and the "rideability "on this section was also good. A typical section of Route 131 is shown in Figure 34.

Another example was found 1.6 km east of Acme on U.S. Route 72. This road was reconstructed in 1982 and the asphalt cement used was 200/250 pen grade. After 5 years of service, no transverse cracking was seen and rutting of approximately 6.4 mm under the wheel path was observed. With respect to cracking in some areas we saw faint scars of transverse cracks. These cracks may have formed during the winter and traffic may have closed it up by some form of kneading action in the summer. A typical section is shown in Figure 35.

Another example was found on Route 32 outside of Hillman. This road was constructed in 1966 with 200/250 pen asphalt. Again very few low temperature cracks were observed and rutting was minimal. A typical section is shown in Figure 36. Thermal cracks were seen along other sections of this route. For example thermal cracks spaced about 61 m apart were seen on Route 115, outside of Cadillac and is shown in Figure 37.

Vermont

Vermont DOT currently specifies AC 10 as the binder for their asphalt concrete pavements. The asphalt specifications are based on a modified version of AASHTO Table 2. The Vermont modification on AASHTO Table 2 is to place a maximum allowable penetration at 25°C of 120. Based on penetration and kinematic viscosities provided by the suppliers in 1987, the PVN of the asphalt was in the range –0.1 to –0.5. Jerd* stated that these were typical values for past years.

Prior to 1978, AC 5 or 150/200 pen asphalt was used as the binder. Most of the AC 5 or 150/200 pavements have now been overlaid with AC 10. The reason for the change to AC 10 was because of flushing. The problem was in the design of the mix, as the asphalt content of the AC 5 was thought to be higher than required.

Vermont requires the coarse aggregates to be crushed and at present there are no crushed re-

*Jerd, C.E., Vermont Dept. of Transportation, personal communication, 1987

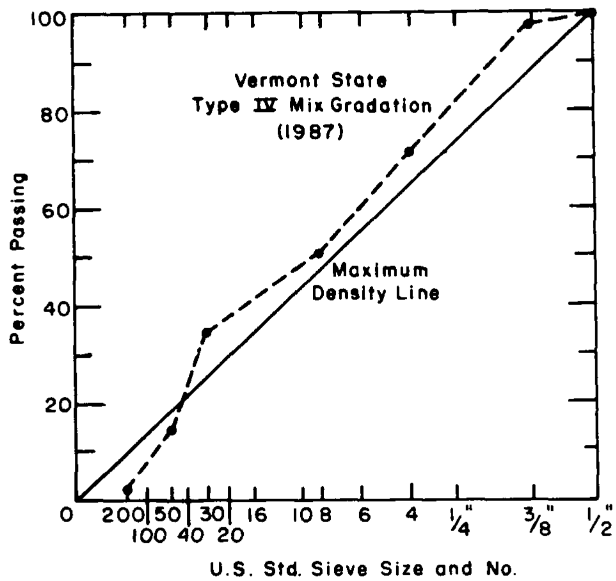

Figure 38. Gradation curve on 0.45 sieve size.

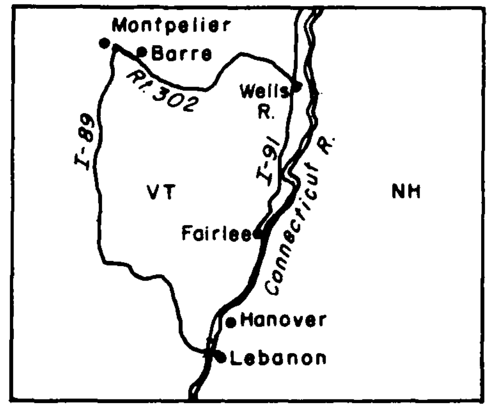

Figure 39. Vermont State survey map.

quirements on the fine portion. At present, Vermont DOT specifications call for 50% natural material so as to increase the workability of the mix. As seen in Figure 38, the gradation of a 12.7 mm maximum size surface course gradation has a hump around the no. 30 sieve on the 0.45 power gradation curve. This hump suggests that the mixture is oversanded and is also a good indicator of a mix susceptible to rutting (Goode and Lufsey 1962). McLeod (1974) showed that the VMA of the mix is increased by oversanding the mix. This hump around the no. 30 sieve would be smaller if, instead of natural sand, manufactured sand is

Figure 40. Low temperature cracks on Route 302 outside Montpelier, Vermont.

Figure 41. Low temperature cracks on Route 5N outside Norwich, Vermont.

Figure 42. Rutting on Interstate 91 near Fairlee, Vermont.

used. McLeod (1974) also showed in the same study, that the mix stability is increased when angular sand in combination with rounded coarse aggregate is used. Jerd* stated that more roads in Vermont are now showing signs of rutting. Some of these roads have performed well for over 10 years and are only now showing signs of rutting. This is possibly due to increased truck tire pressures seen on the highways.

Jerd* pointed out a section on Route 7S in Rutland where the pavement was constructed in the summer of 1987 with AC 20 and had already shown signs of rutting. This shows that changing the asphalt from AC 10 to AC 20 did not control or reduce rutting.

A limited survey (see Fig. 39) of Vermont flexible pavements was conducted and again visual inspection was conducted in a slow moving vehicle. There was minimal low temperature cracks or rutting on Interstate 89 between White River Junction and Montpelier. The section between Fairlee and Newbury on Interstate 91 was constructed in 1977 with 150/200 pen grade asphalt. No thermal cracks were seen but rutting was seen in the right lane. A typical pavement section is shown in Figures 40, 41 and 42.

*Jerd, C.E. Vermont Dept. of Transportation, personal communication, 1987.

Alaska

Alaska DOT specifies AC 2.5 as the asphalt binder in Fairbanks and AC 5 in the Anchorage area for their highway and airfield pavements. The asphalt specifications are based on AASHTO Table 2. Currently, the main supplier of asphalt is MAPCO out of North Pole. Based on information provided by the refinery, the AC 5 has a typical penetration at 25°C of 150 and a kinematic viscosity at 135°C range of 174 to 201 cst. The AC 2.5 has a penetration range at 25°C of 220 to 275 and a kinematic viscosity at 135°C range of 130 to 180. The state has no requirements for temperature susceptibility. However, a typical PVN value for the AC 2.5 and AC 5 asphalt was –0.8.

The state has a crushed requirement on the coarse aggregate (minimum of 70% single face fracture). There is no crushed requirement on the fine portion, and the mix usually contains about 50% natural material. The largest stone in the surface course is 2.54 mm. The Type 1 (used on primary roads) surface course gradation is plotted in Figure 43. This gradation does not have the hump seen in Figure 38 and because of the larger stones used, the mix should be more rut resistant. Many states are removing the hump in the grada-

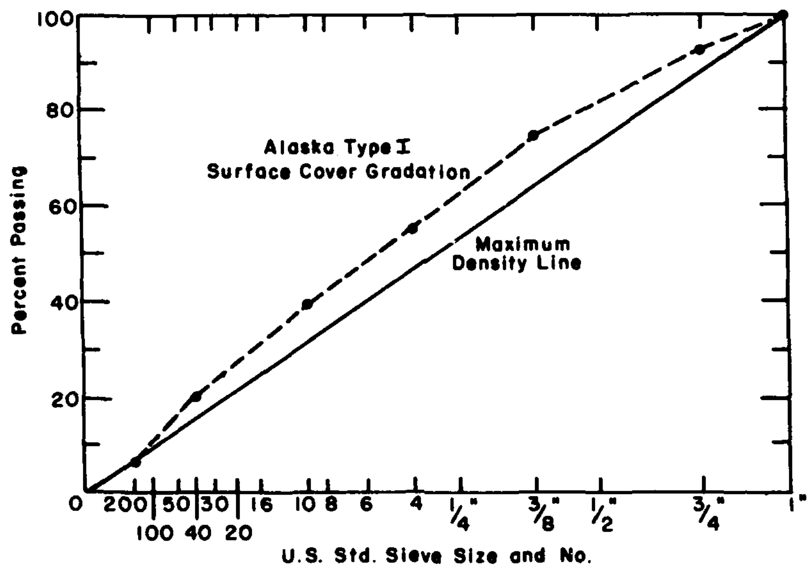

Figure 43. Gradation curve on 0.45 power sieve size opening graph.

tion of their rut resistant mix. The asphalt content is 6% and the design air void is 3% to 5%. The mix is based on Marshall mix design requiring 50 blows for highways and minimum stability of 6.7 kN.

In our survey, the pavements in Fairbanks were badly cracked (winter temperatures of –40°C are common here) but no significant rutting was observed. In Anchorage, approximately 80% of the pavements have significant rutting. Rut depths up to 44 mm were measured at intersections. One other possible cause of this excessive rutting at intersections is the wearing of the surface due to studded tires used during the winter. To try to reduce rutting, the Anchorage DOTPF District experimented with using a harder asphalt. A badly rutted portion of the Seward Highway between 4th and 36th Avenues was leveled and resurfaced with AC 10 in 1987. Some rutting was reported in 1988. Alaska DOTPF is conducting a study to determine the cause of this rutting.

SUMMARY AND CONCLUSIONS

This study looked at the performance of airfield and highway pavements constructed with soft asphalts in cold regions. The study concentrated on the field performance basically in the northern U.S. and Alaska. The issue of rutting and soft asphalts was also addressed.

Briefly, the study found that pavements constructed with soft asphalt binders had a higher resistance to low temperature cracking than harder asphalt binders. It was also shown that the temperature susceptibility of the asphalt had a significant role in reducing low temperature cracking. Higher viscosity asphalts with low temperature susceptibility were reported to perform better than low viscosity, high-temperature susceptible asphalts with respect to low temperature cracking. Studies in Maine have shown that softer asphalts retard reflection cracking and increase the pavement life by at least two years. These studies have also shown that besides asphalt properties, other factors such as subgrade type, aggregate type, air void content, coefficient of thermal contraction of the mix and pavement thickness have an influence on low temperature cracking.

Rutting of asphalt concrete pavements is influenced by the properties of the mixture and not of the asphalt cement alone. All the studies mentioned above strongly recommend using crushed aggregates to reduce rutting and there is a general agreement that the influence of the asphalt cement in a good mix design is negligible. The trend is to characterize the mix with respect to rutting using wheel loading tests on asphalt concrete slabs in the laboratory. Other factors that affect rutting are compaction effort, subgrade preparation, truck tire pressures and type of tire. Many of the ruts seen on highways today are generally due to increased truck tire pressures. The design of intersections should be based on compressive and shear strength of the mix as intersections are subjected to excessive shear stresses.

The use of additives in asphalt cement for the purpose of reducing rutting may be an expensive alternative. Other alternatives that would increase the frictional component of shear strength in asphalt mixtures should be explored, for example, the use of large stones in the mix. The use of additives to increase the tensile strength of an asphalt or asphalt mixture at low temperatures should be studied.

The mixture properties for a rut resistant pavement are not really contrary to those necessary for a thermal crack resistant pavement. A series of surveys of airfield pavements found that cracking was a major problem in northern United States and Canada. All concluded that low temperature cracking was due to the changing stiffness of the asphalt concrete mixture with change in temperature. Other factors that affected crack propagation were pavement thickness, local minimum temperature, asphalt temperature susceptibility and the coefficient of thermal contraction of the mixture. It was interesting to note that rutting was not considered to be a major problem in these surveys.

Most states in the northern U.S. agree that soft asphalts should be used for controlling low temperature cracking. Some of these states have changed to a harder grade asphalt in the last five years because of excessive rutting on their roads, but most have tended to change the aggregate characteristics and maintain the grade of asphalt currently used. Changing to a harder grade without changing the aggregate characteristics provides essentially no relief to rutting. Many states have recognized the need for coarser angular aggregates in their mix for minimizing rutting and are adopting new specifications for crushed aggregates in the mix. However, many states are still using natural sand in their fine proportion so as to increase the workability of the mix. Sometimes this leads to oversanding the mix which then leads to potential rutting. Many states therefore are adopting the use of the 0.45 gradation curve to

check if the gradation of the aggregates is prone to rutting.

Most of the states contacted do not have a temperature susceptibility specification on the asphalt used. Some states indirectly control temperature susceptibility by limiting the maximum penetration of the asphalt and is usually below the Corps temperature susceptibility requirements. With respect to PVN, the asphalts used by the states average around –0.8 and the Corps specifications require a minimum of –0.5.

On the question of using soft asphalt and consequent rutting in airfield or highway pavements, there is no clear evidence that soft asphalt will lead to excessive rutting in the summer. On the contrary, there is evidence to suggest that changing the aggregate characteristics is a better solution to the rutting problem than changing the asphalt grade only. It is also apparent that, in this country, rutting became a major problem in the last five years and it suggests that the mix design be changed to account for the increased loading seen by the pavements. In urban areas most of the ruts are found at highway intersections. Therefore, the mix design in the vicinity of these intersections should be considered as a separate feature from the roadway.

It was found, that the propagation of low temperature cracking is a function of the asphalt and asphalt concrete properties. Currently, the behavior of the asphalt concrete mixture at low temperatures is based on the asphalt rheology at higher temperatures. Research on characterizing the behavior of the mixture at low temperatures should be developed. It is possible to minimize cracking by introducing additives to the asphalt cement or to the asphalt concrete mixture. Research in this area should also be pursued. Field techniques for controlling low temperature cracking are lacking and also need to be studied.

Soft grade asphalts are used in AC mixes in Canada because of the low temperatures in the provinces. A field study on the performance of these asphalts with respect to low temperature and rutting should be conducted. New mix design methods have been introduced in several European countries to reduce rutting and new testing methods and equipment have been developed to characterize the rutting potential of a mix. A study to document European experience and the possibility of implementing the European mechanistic design procedures should be conducted. Research has been conducted in the Scandinavian countries on low temperature cracking and a field study on the performance of asphalt pavements in Scandinavia is warranted.

LITERATURE CITED

AASHTO and FHWA (1987) *Proceedings of Symposium/Workshop on High Pressure Truck Tires, Austin, Texas,* American Association of State Highway Transportation Officers and Federal Highway Administration.

Acott, S.M. (1987) Heavy duty large stone hot mix asphalt. In *Proceedings, 28th Annual Bituminous Paving Conference, University of Illinois at Urbana-Champaign, October 28th.*

Basma, A.A. and K.P. George (1984) Environmental factors in flexible pavement design. In *Design, Evaluation, and Performance of Pavements,* National Research Council, Transportation Research Board, Transportation Research Record 954, p. 52–58.

Betenson, W.B., D.I. Hanson, W.C. Jones, J.J. Maykuth, R.T. Rask, R.D. Tea and R.G. Warburton (1984) Asphalt pavement rutting. Western Association of State Highways and Transportation Officials, Report No. FHWA/TS-84/211.

Bissada, A. (1983) Compactability of asphalt paving mixtures and relation to permanent deformation. National Research Council, Transportation Research Record 911.

Brown, R. (1987) Tire pressure and its relationship to rutting. Panel Discussion, 63rd Annual Open Meeting, Northeastern States Engineers Association, Burlington, Vermont, Oct.

Burgess, R.A., O. Kopvillem and F.D. Young (1971) St. Anne test road—Relationships between predicted fracture temperatures and low temperature field performance. In *Proceedings, Association of Asphalt Paving Technologists,* **40:** 148–193.

Carpenter, S.H. and R.L. Lytton (1972) Procedure for predicting occurrence and spacing of thermal susceptibility cracking in flexible pavements. CFHR, University of Texas, Report No. 123-14.

Carpenter, S.H. and T. VanDam (1985) Evaluation of low-temperature performance of asphalt cements. USA Cold Regions Research and Engineering Laboratory, Unpublished Report.

Davis, R.L. (1987a) Relationship between the rheological properties of asphalt and the rheological properties of mixtures and pavements. In *Asphalt Rheology: Relationship to Mixture* (O.E. Briscoe, Ed.)., American Society of Testing Materials STP 941, p. 28–51.

Davis, R.L. (1987b) Resolution of modern hot mix

asphalt problems through historical insights. Unpublished draft report.
Deme, I. J. and F.D. Young (1987) Ste. Anne test road revisited twenty years later. In *Proceedings, Canadian Technical Asphalt Association*, **32:** 254–283.
Epps, J.A., B.M. Gallaway and W. Scott, Jr. (1970) Long-term compaction of asphalt concrete pavements. Highway Research Record No. 313.
Fabb, T.R.J. (1974) The influence of mix composition, binder properties and cooling rate on asphalt cracking at low temperatures. In *Proceedings, Association of Asphalt Paving Technologists*, **43:** 285–331.
Finn, F.N. (1963) A review of the AASHTO road test with interpretation and applications. Paper presented at Centre d'Information du Bitume Symposium, Brussels, Belgium.
Finn, F.N., K. Nair and J. Hilliard (1976) Minimizing premature cracking of asphalt concrete pavements. National Cooperative Research Program Report No. 195, 1978.
Ford, M.C. (1985) Development of a rational mix design method for asphalt bases and characteristic of Arkansas asphalt mixtures. Report Federal Highway Administration/AR-85/004.
Foster, W.T. (1987) Comparison of service life to PSI (separated by total 18K and asphalt type). Inter-Departmental Memorandum, State of Maine.
Gaw, W.J. (1978)The measurement and prediction of asphalt stiffnesses at low and intermediate pavement service temperatures. In *Proceedings, Association of Asphalt Paving Technologists*, **47:** 457–494.
Goetz, W.H., J.F. McLaughlin and L.E. Wood (1957) Load-deformation characteristics of bituminous mixtures under various conditions of loading. In *Proceedings, Association of Asphalt Paving Technologists*, 26, 237.
Goode, F.J. and L.A. Lufsey (1962) A new graphical chart for evaluating aggregate gradations. In *Proceedings, Association of Asphalt Paving Technologists*, **31.**
Haas, R., H. Lee, F. Meyer and G. Argue (1987) Cold climate performance of Canadian airport pavements. In *Proceedings, Paving in Cold Areas*, Vol. 1.
Henry, J.W. (1981) Pavement problems in Alaska versus asphalt grading. Report, FHWA-AK-82-8, Alaska DOTPF.
Heukelom, W. and A.J.G. Klomp (1964) Road design and dynamic loading. In *Proceedings, Association of Asphalt Paving Technologists*, **33:** 92–125.
Hofstra, A. and A.J.G. Klomp (1972) Permanent deformation of flexible pavements under simulated road traffic conditions. In *Proceedings, 3rd International Conference on the Structural Design of Asphalt Pavements*, **1:** 63–621.
Huber, G.A. and G.H. Heiman (1987) Rutting of Asphalt Pavements in Saskatchewan. In *Proceedings, Canadian Technical Asphalt Association*, 32:152–176.
Kallas, B.F. (1982) Low temperature mechanical properties of asphalt concrete. The Asphalt Institute, Research Report 82-3.
Lefebvre, J.A. (1970) A modified penetration index for Canadian asphalts. In *Proceedings, Association of Asphalt Paving Technologists*, **39:** 443–480.
Lister, N.W. (1972) The transient and long-term performance of pavements in relation to temperature. In *Proceedings, Third International Conference on Structural Design of Asphalt Pavements, London*, p. 94–100.
McLeod, N.W. (1969) Discussion of paper on Ste. Anne test road. In *Proceedings, Canadian Technical Asphalt Association*, **14:** 110–131.
McLeod, N. W. (1972) A 4-year survey of low temperature transverse pavement cracking on three Ontario test roads. In *Proceedings, Association of Asphalt Paving Technologists*, **41:** 424–495.
McLeod, N. W. (1974) A laboratory investigation of the compaction of dense-graded asphalt concrete. In *Proceedings, Canadian Technical Asphalt Association*, **19:** 377–450.
McLeod, N.W. (1976) Asphalt cements: Pen-Vis number and its application to moduli of stiffness. *ASTM Journal of Testing and Evaluation*, **4(4).**
McLeod, N.W. (1978) Test data from three Ontario test roads after 15 years of service. In *Proceedings, Canadian Technical Asphalt Association*, **23:** 534–593.
McLeod, N.W. (1979) Asphalt cement specifications—Some facts and challenges. Presentation, American Association of State Highway and Transportation Officials.
Morris, J. and R.C.G. Haas (1972) Designing for rutting in asphalt pavements. In *Proceedings of the Seventeenth Annual Conference of Canadian Technical Asphalt Association*, **17:** 169–183.
Michigan State Department of Transportation (1987) Roadway—Bituminous Pavement Design Guidelines. Unpublished draft report.
Novak, Jr., E.C. (1976) Evaluation of a model for predicting transverse cracking of flexible pavement. Michigan Department of Transportation, Research Report No. R-975.
Osterkamp, T.E., G.C. Barker, B.T. Hamer, J.P. Gosink, J.K. Peterson and V. Gruol (1986) Low temperature transverse cracks in asphalt pavements in interior Alaska. Report, Alaska Department of Transportation and Public Facilities.

Palmer, R.K. and J.J. Thomas (1968) Pavement density—How it changes. In *Proceedings, Association of Asphalt Paving Technologists,* **37**: 547–583.

Pfeiffer, J. Ph. and Van Doormal (1936) The rheological properties of asphaltic bitumens. *Journal of the Institute of Petroleum Technologists,* **22.**

Pilsbury, A. and K.M. Jacobs (1985) The performance of bituminous pavement as it relates to the use of viscosity grade AC–5 vs. AC–10 asphalt cement. Interim Report, Maine Department of Transportation.

Seed, H.B., C.K. Chan and C.L. Monosmith (1955) Effects of repeated loading on the strength and deformation of compacted clay. In *Proceedings, Highway Research Board,* p. 541–558.

Seed, H.B., C.K. Chan and C.E. Lee (1962) Resilience characteristics of subgrade soils and their relationship to fatigue failures in asphalt pavements. In *Proceedings 1st International Conference on the Structural Design of Asphalt Pavements,* p. 611–636.

Shahin, M.Y. and B.F. McCullough (1972) Prediction of low temperature and thermal fatigue cracking in flexible pavements. CHFR, University of Texas, Report No. 123-14, August.

Sugawara, T., H. Kubo and A. Moriyoshi (1982) Low temperature cracking of asphalt pavements. In *Proceedings, Paving in Cold Areas, Mini-Workshop, Canada/Japan Science and Technology Consultations,* **1**: 1–42.

Sugawara, T. and A. Moriyoshi (1984) Thermal fracture of bituminous mixtures. In *Proceedings, Paving in Cold Areas Mini-Workshop, Canada/Japan Science and Technology Consultations,* **1**:291–319.

Uge, P. and P. Van De Loo (1974) Permanent deformation of asphalt mixes. *Canadian Technical Asphalt Association.* **29**: 307–342.

Van der Poel, C. (1954) General system describing the visco-elastic properties of bitumens and its relation to routine test data. *Journal of Applied Chemistry,* **4**: 221.

Vesic, A.S. and L. Domaschuk (1964) Theoretical analysis of structural behavior of road tests flexible pavements. Highway Research Board, NCHRP Report 10.

Vinson T.S., R.L. Berg and H. Tomita (1987) Statement of research needs to address airport pavement distress in cold regions. In *Proceedings, Paving in Cold Areas,* **2**: 981–1012.

Warren, F.J. (1901) Pavement of roadway. U.S. Patent 675430, June 4th 1901.

Warren, F.J. (1903) *Pavement.* U.S. Patent 727505, May 5th 1903.

Wyoming State Highway Department, Materials Laboratory (1982) Rutting investigation, Cheyenne, Wyoming, April.

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1. AGENCY USE ONLY (Leave blank)	2. REPORT DATE	3. REPORT TYPE AND DATES COVERED
	May 1990	

4. TITLE AND SUBTITLE	5. FUNDING NUMBERS
Use of Soft Grade Asphalts in Airfields and Highway Pavements in Cold Regions	PE: 6.27.30A PR: 4A762784AT42 TA: BS WU: 003 Interagency Agreement DTFA-01-84-2-02038
6. AUTHORS Vincent Janoo	

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)	8. PERFORMING ORGANIZATION REPORT NUMBER
U.S. Army Cold Regions Research and Engineering Laboratory 72 Lyme Road Hanover, New Hampshire 03755-1290	Special Report 90-12

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)		10. SPONSORING/MONITORING AGENCY REPORT NUMBER
U.S. Army Office of the Chief of Engineers Washington, D.C. 20314-1000	U.S. Department of Transportation Federal Aviation Administration Research and Development Service Washington, D.C. 20491	DOT/FAA/RD-90/12

11. SUPPLEMENTARY NOTES

12a. DISTRIBUTION/AVAILABILITY STATEMENT	12b. DISTRIBUTION CODE
Approved for public release; distribution is unlimited. Available from NTIS, Springfield, Virginia 22161.	(dcg)

13. ABSTRACT (*Maximum 200 words*)

Soft grades of asphalt cement are being used for controlling low temperature cracking in some parts of the northern regions of the United States and in Canada. The U.S. Army Corps of Engineers (COE) specified softer asphalts for use in cold regions (ETL 1110-3-369) dated November 1976; at present, the COE uses the penetration viscosity number (PVN) as a measure of the temperature susceptibility of the asphalt. A minimum PVN of –0.5 is specified for moderately cold areas and –0.2 in regions where the design freezing index is greater than 39900°C-hr. Field studies have been conducted that clearly show the benefits of using softer grades of asphalt for minimizing low temperature cracking in cold regions; however, field studies relating rutting to asphalt type are rare. A major concern is whether or not pavements constructed with softer grades of asphalt are more susceptible to rutting during the hot summer months. A field study was conducted by CRREL to gather information on the use of soft grades of asphalt (AC 2.5, AC 5 and AC 10) and their associated pavement performance. An attempt was made to compare the COE specifications with State DOT specifications for these soft grades of asphalt. The influence of the asphalts studied, and the preliminary results of this field program are presented in this report. For the longer term objectives of this study new or reconstructed pavements in various parts of the country will be monitored for both low temperature cracking and rutting. (KR)

14. SUBJECT TERMS			15. NUMBER OF PAGES 54
Asphalt concrete Cold regions pavements	Pavement cracking Pavement rutting	Soft grade asphalts	16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED	UNCLASSIFIED	UL

NSN 7540-01-280-5500

☆U. S. GOVERNMENT PRINTING OFFICE: 1990--700-057--22013

Standard Form 298 (Rev. 2-89)
Prescribed by ANSI Std. Z39-18
298-102